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A COMPARATIVE STUDY OF THE

PERFORMANCE OF PILES AND PILE FORMULAS

A THESIS

Presented to

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by

Willem Jan van Reenen, Jr.

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A COMPARATIVE STUDY OF THE

PERFORMANCE OF PILES AND PILE FORMULAS

Approved:

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SUMMARY

This thesis correlates and explains data gathered from a test pile program at Barry Steam Plant near Mobile, Alabama in terms of soil mechanics theory.

The program was conducted to determine the pile type best suited to provide foundation support for a new generating unit for the plant. Three types of piles--Raymond step-taper, Hel Cor, and closed-end-pipe--were tested. The piles were jetted and driven to varying depths in inundated firm to dense sands. Data obtained from the driving and load testing of a total of 15 test piles was used to evaluate and compare the accuracy of the Hiley formula and the Engineering News formula for determining pile capacity. Also, data available from soil test borings made in the immediate vicinity of the test piles was used in static analyses of the test piles.

An attempt was made to analyze the effect of jetting on pile capacity. Data obtained from a test pile program conducted at the Jack Watson Steam Plant near Biloxi, Mississippi in similar soils was incorporated into the analyses. The method of jetting used on the single type of pile (timber) was varied.

The comparison of the Hiley and Engineering News formulas showed that the Hiley formula was more accurate in estimating the actual pile capacity. The use of a safety factor

of 2.5 would have been adequate. The capacities predicted by the Engineering News formula varied from one to two times the actual pile capacity even though a theoretical safety factor of six is included in this formula. There was no relationship between the two formulas when applied to the results of the Barry Steam Plant program, but when applied to the Jack Watson Steam Plant program, a definite relationship emerged.

A detailed analysis of portions of the Hiley formula cast doubt upon the commonly used method of field determination of the elastic rebound of pile and soil ($C_2 + C_3$), and the empirical coefficients used to estimate the elastic rebound of the pile (C_2).

A static analysis of pile capacity using Meyerhof's coefficients for deep foundations yielded theoretical pile capacities so high that an alternative was sought. Methods of analysis presented by Berezantsev and Vesic yielded much more reasonable results, with that of Vesic most nearly approximating actual pile performance.

Data from the Jack Watson Steam Plant indicated that piles driven without jetting have greater bearing capacity than jetted piles driven to the same driving resistance, even though the jetted piles may penetrate far deeper.

At Barry Steam Plant, all piles were driven to a driving resistance indicated by the Hiley formula (S.F.=2), with most of the piles being jetted. The Hel Cor pile provided greater capacity with less penetration than the Raymond step-taper piles. The pipe piles drove longer and provided less capacity than either of the other pile types.

CHAPTER I

INTRODUCTION

Although a great deal of research has been done concerning driven piles, the presently available methods for determining pile bearing capacity still contain numerous inaccuracies and unanswered questions. Two separate approaches are commonly used. "Dynamic" analyses make use of the concept that energy input equals the energy output less internal losses. "Static" analyses use soil strength parameters to determine the static forces acting on an imbedded pile.

The dynamic formulas either suffer from a lack of consideration of important factors which influence pile capacity (Engineering News formula) or provide methods for defining these factors which are vague, technically unsound, or rely largely on the personal judgment of the designer (Hiley formula). The "static" methods of analysis while often seeming to be straightforward and exact are also subject to severe limitations, with perhaps the most serious being that the designer must rely entirely upon data obtained from soil borings and laboratory tests for his analysis. Further, recent work by Berezantsev (14) and Vesic (13, 41, 42) has cast doubt on the validity of some of the previously established principles involved in the static determination of pile capacity.

Little can be done toward increasing our knowledge of pile behavior without extensive, full-scale pile testing programs to develop new relationships and to corroborate the results of theoretical research. As test pile programs are quite expensive, the number of piles incorporated into them is generally quite small; seldom exceeding two or three piles. These small programs offer only limited opportunities for determining definite relationships between pile performance and soil mechanics theory. Therefore, when a test pile program containing a large number of test piles plus complete soil boring data is performed, it provides an unusual opportunity for research.

Such a program was performed by Alabama Power Company at the site of a proposed new generating unit at Barry Steam Plant near Mobile, Alabama. Three types of piles; Raymond step-taper, Hel Cor, and pipe (a total of 15 piles), were driven and load tested under the supervision of representatives of Alabama Power Company and Law Engineering Testing Company.

The purpose of the test pile program was to determine the type of pile best suited to withstand the very heavy loadings of the steam generating unit without exceeding the extremely small settlement tolerances for the steam-driven turbine.

As the Raymond and Hel Cor piles were jetted, not only could the performance of the different types of piles be evaluated and compared, but the effect of jetting could be studied. Also, the complete soil test boring data enabled static analyses to be performed and compared with dynamic analyses and actual pile performance. These analyses and comparisons provided worthwhile information as to comparative performance of the methods of analysis as well as that of the piles themselves.

As more owners are made to realize the monetary worth of this type of program and as more engineers lend themselves to the analysis of the results, pile foundations will becomes less and less a mystery.

CHAPTER II

DEVELOPMENT OF THE SUBJECT

Piles And Pile Driving Equipment

"Piles are older than history" (2). Twelve thousand years ago inhabitants of Switzerland built their homes on wooden poles driven into the soft bottoms of shallow lakes, thereby protecting them from the attacks of animals and unfriendly neighbors (2). Venice was built on wood piling in the delta of the Po River. When the Campanile, one of Venice's most famous structures, collapsed in 1902, the submerged foundation piles, driven in 900 A.D. were found to be in good shape and were reused. Even Caesar realized the value of piles, as he used them for the foundation of a bridge which he built across the Rhine River (30).

In those times, piles were driven by primitive methods which consisted of hand mauls, hand operated machine mauls, ratchet winch rams, treadmill drivers, water wheel drivers, and gang operated rams. These primitive pile driving methods contrast sharply with the advanced methods used today. Today's pile driver is generally a moderately high capacity crawler or truck mounted crane. Attached to the boom of the crane are two fixed parallel channels called "leads" which serve as guides for the hammer. The lower portion of the leads is fixed to the crane by a "stay", which is generally adjustable to allow tilting of the leads (2).

The most commonly used type of pile hammer is the steam or air hammer. These hammers use steam or compressed air to raise the hammer and, in the case of double-acting or differential hammers, to apply additional downward thrust. Nasmith invented the steam hammer (single-acting) in 1845, thereby initiating the modern pile driving practices that we know today (30). Single-acting hammers have undergone few changes since their conception. However, subsequent developments have produced the double-acting and differential steam hammers, open and closed-cylinder diesel hammers, and most recently, the sonic or vibratory pile drivers.

Methods of Predicting Capacity

Modern literature on piles dates from an article entitled "Piles and Pile Driving" appearing in Engineering News in 1893. The "Engineering News Formula", which is still the most widely used pile driving formula was published in this paper (30). Since, many other formulas have been developed, all of which fall into four general types: empirical formulas; static formulas; dynamic formulas; and formulas based on the theory of longitudinal impact on a rod (1).

The various empirical formulas which have been proposed generally are based on the results of tests of limited extent and are, therefore, seldom used today.

Static Analyses

Many static analyses have been proposed with some having been widely accepted and used. Some of these formulas are entirely theoretical, while others are derived by an empirical approach. Perhaps the most commonly used static formula analyzes the tip bearing capacity and the skin friction capacity of the pile separately. The use of this method is encouraged by Sowers (2). For determination of the tip bearing capacity, the general bearing capacity equation (31, 32) is used incorporating Meyerhof's constants for deep foundations (3,4). The friction resistance along the pile shaft is determined by summing the resistance provided by each of the soil strata penetrated by the pile. The friction values for different soils, as well as the lateral earth pressure coefficients which should be used are determined from laboratory tests or from published tables (2).

Chellis (1) recommends the use of semiempirical formulas derived by Terzaghi and Peck (32) with elaborations by Meyerhof (4) for the static analysis of piles (1). These formulas consist of the general bearing capacity equation modified by empirical constants. Meyerhof extended them to include the effect of compaction during pile driving. As the method of analysis described by Sowers is more common, it has been used for analysis in this paper. It is discussed in more detail on succeeding pages.

Dynamic Analyses

The dynamic formulas that have been proposed are many, and the difference between their results is often large. They are based on the assumption that the capacity of the pile is equal to the dynamic driving energy less internal losses within the system. The basic principle is that the energy input (weight of ram multiplied by its fall) is equal to the energy output (driving resistance multiplied by the set of the tip of the pile). Several formulas simply consisting of this equation plus a safety factor have been proposed (1).

There are many factors which affect the above mentioned principle which are not included in the basic equation. The most notable of these is energy loss within the system. Other formulas have been developed which attempt to allow for the presence of these factors. The simplest of these formulas use a coefficient to compensate for the presence of the energy loss factors. Included in this group are the Engineering News, Wellington, Vulcan, and Bureau of Yards and Docks formulas (1).

The Dutch, Ritter, and Benabencq formulas form a group which attempts to expand the coefficients by incorporating expressions for the efficiency of applied energy by including the relative weights of pile and hammer (1).

The most detailed dynamic formulas now used form the final group. These include the Rankine (1), Redtenbacher (36), Hiley (37) and Schenk (38) formulas (1). These formulas contain a series of terms designed to allow for energy loss during driving. A detailed comparison of all of these formulas is beyond the scope of this paper. However, the reader is encouraged to consult papers by various writers which do make this comparison (11, 16, 27, 33, 34).

The most commonly used dynamic formula in this country is still the Engineering News formula. However, the Hiley type of formula is being used more and more. The Hiley formula and the Engineering News formula have been used in the analysis of the results of the test pile program which provided the data for this paper and will be discussed in detail on the following pages.

The last group of equations are the so-called "wave equations". The wave equations themselves are fairly old, but have only recently been applied to piles. These equations, which are based on the theory of longitudinal impact on a rod, were developed by De St. Venant and Boussinesq (1). When the hammer strikes the pile cap, a force is transmitted through the cap to the top of the pile. The force accelerates the top of the pile and is transmitted progressively downward causing the pile to behave like a spring. The force wave is partially dissipated in overcoming skin friction as it progresses down the pile. The remainder is dissipated in overcoming end bearing at the tip. The shape of the wave depends on the pile rigidity, and the peak force depends on hammer energy and efficiency.

The general wave equation provides results which compare favorably with those provided by the Hiley formula, except that the Hiley formula may tend to underestimate the capacities of long and heavy piles (1). A shortcoming of the wave equations is that "the general wave equation ... is, when complicated by actions of ram, cap block, pile, and ground, too complex for manual solution"(1). Electronic computers must be used for solution. Further, they have limited use in evaluating pile capacity because the effects of skin friction and end bearing are difficult to evaluate under field conditions and cannot be predicted in advance.

The preceding information was not intended to provide complete information on the history and development of piles and pile formulas. It was, instead, intended to briefly introduce the reader to pile driving history, equipment, and analysis so that he might be better prepared for the pages which follow. More complete information is available in the noted references which are contained in the bibliography.

CHAPTER III

AREA GEOLOGY AND SOIL CONDITIONS

Barry Steam Plant is located in Bucks, Alabama, approximately 20 miles north of Mobile, Alabama. It is underlain by terrace and alluvial deposits in the flood plain of the Mobile River (27). Hills in the area are often capped by sand, gravel, and lenticular white to variegated clay of Citronelle formation of Pliocene or early Pleistocene age. The Citronelle formation is underlain by deposits of the Miocene age consisting of sandstones, variegated argillaceous sand, and dark variegated clay. In Bucks and in Mobile, the Mobile River has eroded the Citronelle formation so that the alluvial deposits generally lie on sediments of the Miocene age (27).

The Citronelle formation is predominantly sandy, but contains lenses of clay. The sands are cross-bedded and generally red. The clays vary in color according to the extent to which they are weathered and are mottled gray and purple, red, or yellow, with pebbles or pellets being common. There are also extensive gravel deposits in the Citronelle, ranging in thickness from a few inches to 340 feet (29).

The upper part of the alluvial deposits often consists of decayed trees, vegetal matter, and carbonaceous clay. The clay grades downward into fine argillaceous sand, coarse sand, and gravel, interbedded with lenticular clay beds. The lower beds of the alluvium, generally of coarse gravel, overlie the Miocene deposits (27). The alluvial deposits in the Mobile area are generally 80 to 150 feet thick.

The Miocene deposits, which extend to a depth of 1300 to 1400 feet consist of grey clay, sandy clay, fine argillaceous sand, and medium to coarse sand, with a gravelly sand about 300 feet thick at the base. The sand and clay beds are highly lenticular. The Miocene deposits have a regional dip slightly west of south in Mobile County, and at surface exposures in the outcrop area north of Mobile, they appear to dip 15 to 25 feet per mile (27).

The borings made for Unit 4, Barry Steam Plant encountered 105 feet of alluvial deposits before entering firmer soils, probably sediments of the Miocene age. Generally, the surface soils consisted of from 4 to 20 feet of stiff brown and gray sandy clay containing some sand lenses and firm clayey sand deposits. In the area of the boiler room these materials are underlain by dense to very dense, white, tan, and brown sands which contain some rounded gravel. The turbine room is also underlain by gravelly sand, but the sand is generally firm in consistency. Fingers of the firm sand extend into the dense sand under the boiler room. The firm sand contains a number of clay lenses, the most notable of which is a 3 to 8 foot thick deposit of soft clay which underlies a portion of the turbine room at a depth of about 50 feet.

The recent alluvial deposits end at elevation -85. Below them is a stiff to very stiff blue clay of the Miocene age. Underlying this clay is firm to dense grey sand which is sometimes clayey and contains lenses of stiff to very stiff blue clay. A typical soil profile is shown in Figure 1.

Ground water was encountered at a depth of approximately 19 feet (elevation +1). However, because the Mobile River is very close to the site and because the soils of the site are predominantly sands of high permeability, the ground water level will be subject to large and rapid fluctuations with changes in the level of the river.

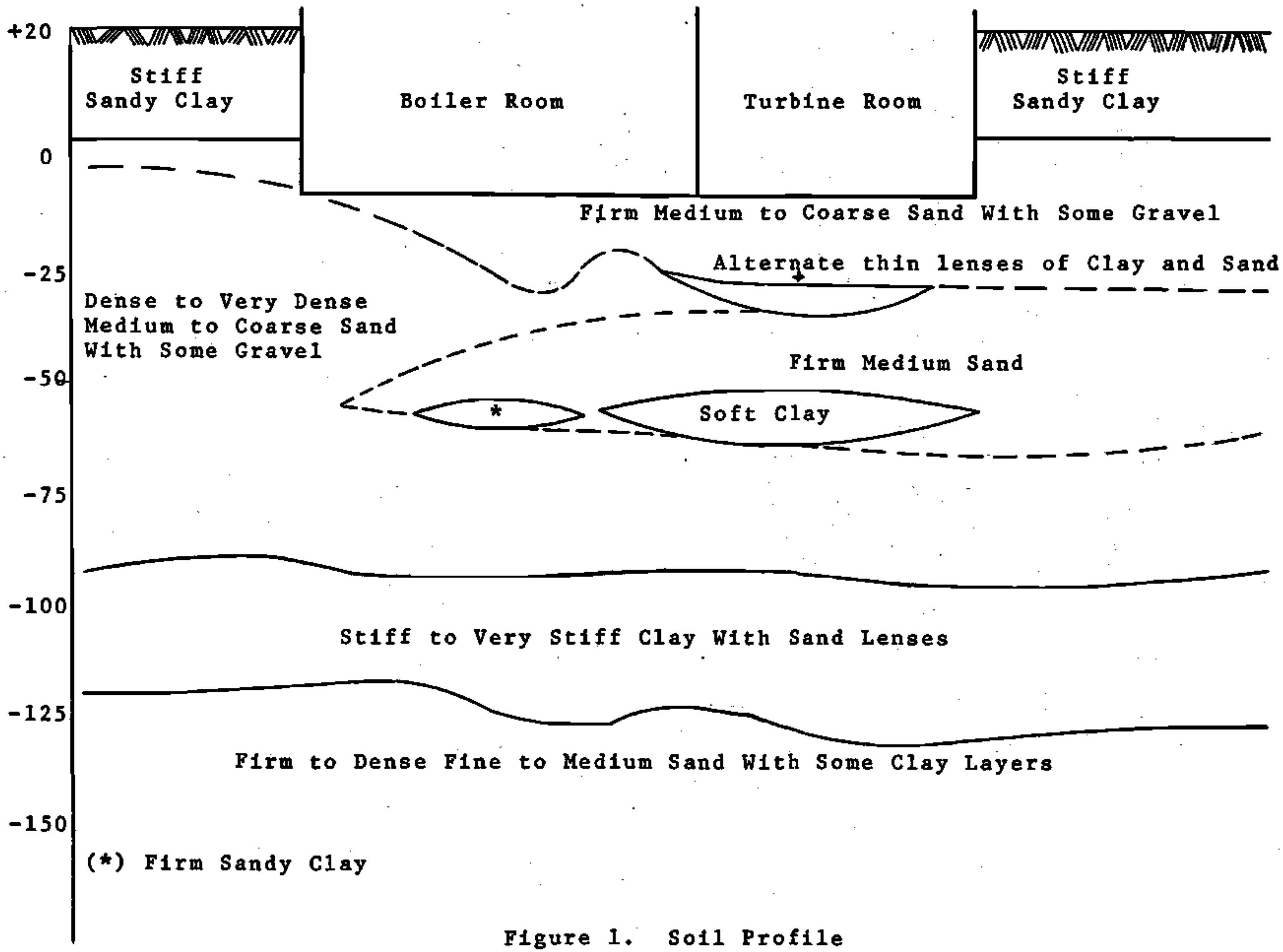


Figure 1. Soil Profile

CHAPTER IV

EQUIPMENT AND PROCEDURE

Pile Driving

Gulf City Construction Company drove and tested the piles under the direction of Alabama Power Company and Law Engineering Testing Company.

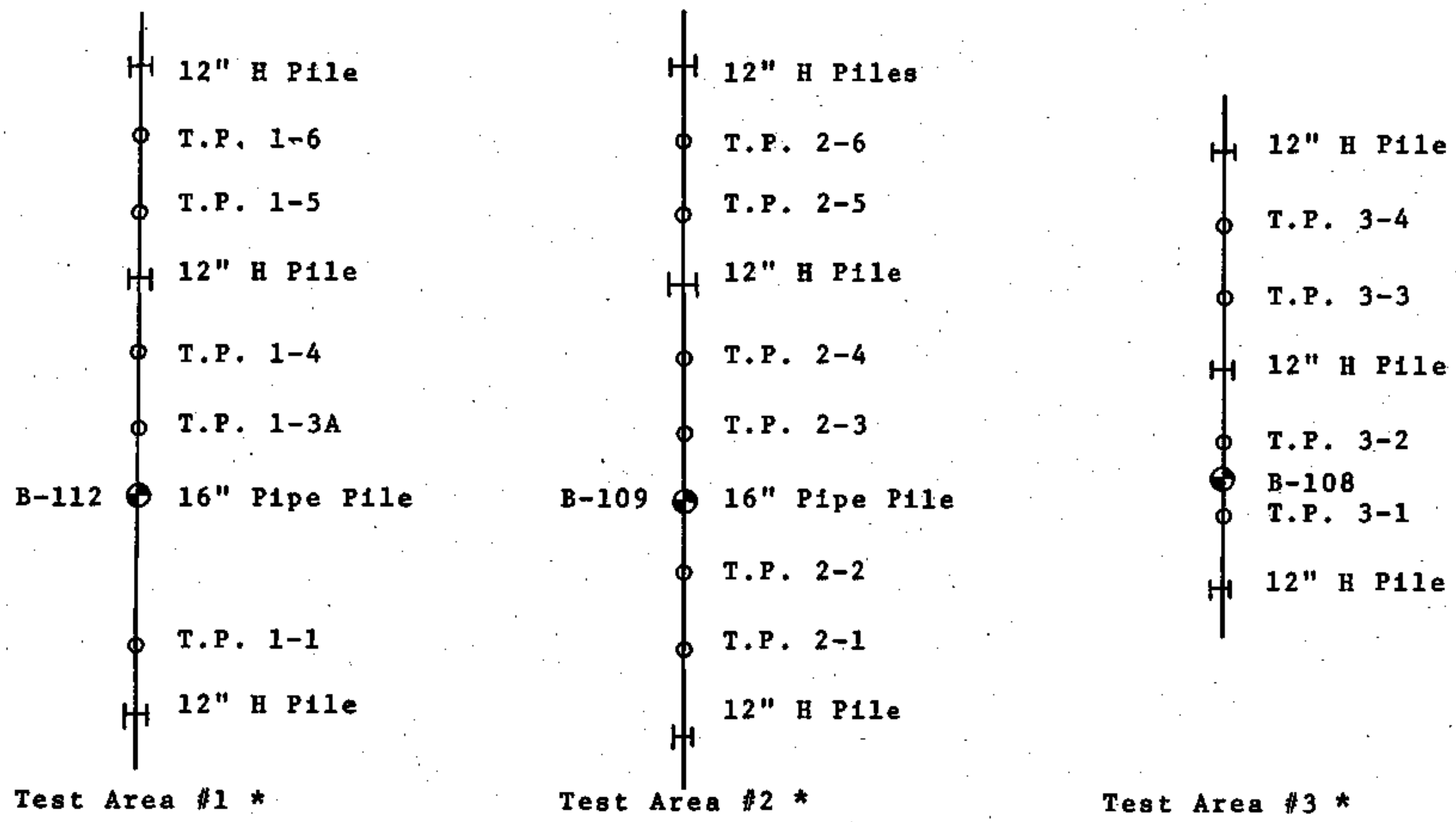
The piles were driven with a Vulcan 06 single acting hammer having a ram weight of 6500 lbs. and a fall of 36 inches. Air pressure was supplied to the hammer by two 600 c.f.s. compressors (one Gardner-Denver and one Ingersoll-Rand) feeding into a central reservoir. These compressors provided the 100 psi air pressure required to operate the hammer at its design speed of 60 blows per minute. The cushion block was "Ascon", a laminated asbestos cushion developed by Vulcan Iron Works, which has a coefficient of restitution of approximately 0.55 according to the manufacturer.

The jets consisted of two 4 1/2 inch I.D. pipes with 2 inch nozzle openings, and four 1/2 to 5/8 inch side ports located about six inches above the nozzle opening. Water was supplied by two Gorman-Rupp Model 54J2-B centrifugal pumps, each capable of supplying from 100 to 800 g.p.m. at 220 to 75 p.s.i.

Two lengths of each pile type were driven. The short Raymond pile was a 00BR having a 9 1/2 inch tip diameter, seven eight foot sections, and a 16 3/8 inch butt diameter. The long Raymond pile had a tip diameter of 8 5/8 inches, three eight foot sections, five twelve foot sections, and a butt diameter of 15 3/8 inches. The Hel Cor piles were 80 feet and 50 feet long and were driven with the same 80 foot Vulcan (mechanically expanding) mandrel. The shells of both the Raymond and Hel Cor pile were of 14 gage corrugated steel.

The pipe piles were driven in 60 foot and 80 foot lengths without an internal mandrel. They were 12 3/4 inch O.D. pipe with a wall thickness of 0.25 inches. The base plate was 5/8 inches thick.

At the time of the load test program, the site had been graded to elevation +4, which was as deep as excavation could progress due to the water table. As the job piles were to be driven from elevation -8, the test piles were driven through steel casings which extended from the ground surface to elevation -8. These casings were driven into the ground and then cleaned out to elevation -8 to simulate actual job conditions. The piles were driven in three groups as shown in Figure 2. In Groups #1 and #2, two piles of each type were driven, attempting to drive one each to tip elevation -75 and one each to tip elevation -45. In Group #3, one Raymond, one Hel Cor, and two pipe piles were driven attempting to drive all four to tip elevation -50.



* Piles spaced 4 feet center to center.

Figure 2. Test Pile Locations - Barry Steam Plant

The long Raymond and Hel Cor piles were prejetted to near elevation -50 and then driven with the aid of a side jet to near the prescribed tip elevation. The short Raymond and Hel Cor piles were prejetted to approximately elevation -30 and driven to near elevation -45. The long pipe piles were prejetted to elevation -35 and driven to approximately elevation -75, while the short pipe piles were prejetted to elevation -8 and driven to approximately elevation -45.

In addition to the prescribed tip elevations, it was specified that no piles be stopped before they had met the driving criteria for 40 ton piles as established by the Hiley formula (S.F.=2). Data supplied by the pile hammer manufacturer and the pile supplier was combined with empirical constants presented by Chellis (1) to determine the driving criteria by the Hiley formula.

Load Testing

The loading of the test piles was accomplished by the use of hydraulic jacks reacting against heavy H-beams attached to reaction piles. Test pile deflection was measured with two micrometer dial gages, reading to 0.001 inches, mounted on a 12 foot long wood beam. This beam, in most cases, was attached securely to two of the other test piles in the same group. When this was not possible, it was attached at one end to a stake driven into the ground and at the other end to a test pile. The dial gages were mounted on this beam and the plunger brought to bear on pieces of angle iron welded to the pile.

Scales with mirrors were placed on the test pile and the two principal reaction piles to provide an independent check on test pile movement and to allow observation of the behavior of the reaction piles. The scales were read by means of a wire stretched between two stakes 12 feet apart.

Load was applied to the piles in 10 ton increments beginning at 10 tons and continuing to 30 tons, then at 5 ton intervals to 50 tons, then at 10 ton intervals to the failure load or until a load of 100 tons was attained. Unloading was carried out in 20 ton decrements from 100 tons.

In accordance with the job specifications, settlement of the pile under each load was measured at the following times after the load was applied: 1, 2, 5, 10, 15, 30 minutes and at 15 minute intervals thereafter until the settlement was less than 0.0005 inch in 15 minutes. The testing was considered complete when 100 tons was reached or when the pile failed. Failure was defined as the load at which the rate of settlement of the test pile exceeded 0.05 inches per ton of additional load, or when the total settlement of the pile reached 1 1/2 inches.

A load vs settlement curve was prepared for each pile. These curves are contained in the Appendix. The shape of the load vs. settlement curves for Test Piles 1-1, 1-5, and 3-1 indicates that the piles were approaching failure at the maximum load. For these piles a failure load was estimated by extending the load vs. settlement curves.

Analysis of Results

Three basic methods of pile capacity determination were used to analyze the data obtained from the test pile program. These were: 1) static analysis, 2) Hiley formula, and 3) Engineering News formula.

Static Analysis

The comprehensive subsurface investigation performed at the site provided data for use in a static analysis of pile capacity. For analysis, the soil type and standard penetration resistance were determined from borings made in the center of each load test area. For cohesionless soils, the standard penetration resistances were compared with published relationships between penetration resistance and angle of internal friction (40) to obtain the angle of internal friction for each stratum. These values were compared with fragmentary laboratory tests performed on undisturbed samples obtained from borings made at the site and other published relationships (1) for verification. The friction between the soil and pile for clay strata was obtained from a published table relating standard penetration resistance to friction value (1). In analyzing the corrugated piles (Raymond and

Hel Cor) in sand, the friction angle between pile and soil was assumed to be equal to the angle of internal friction of the sand (2). For the smooth pipe piles, the coefficient of friction was taken to be between 0.2 to 0.4 from published estimates presented by Sowers (2). The exact values used for each stratum penetrated by the pile was interpolated between these limits depending on the angle of internal friction of the stratum. The specific values are contained in the Appendix.

The effective overburden stress at the center of each stratum analyzed was determined by multiplying the effective unit weight of the soils by the depth below the ground surface. The coefficient of effective lateral pressure of the soils on the jetted, corrugated piles was assumed to be 1.0. This value is presented by Sowers (2) for use with piles driven without a jet into medium consistency sands. The skin friction for each stratum penetrated by the piles was determined by multiplying the effective overburden stress by the coefficient of lateral earth pressure and by the coefficient of friction of the material composing the stratum. The total skin friction was obtained by summing the friction values of the soil strata penetrated.

The general bearing capacity equation was used to determine the ultimate end bearing of the piles. Bearing capacity factors developed by Meyerhof (3) for deep foundations were used along with the values of overburden stress calculated as previously described. This entire method of analysis is described by Sowers.(2). Hereafter it will be referred to as the "Meyerhof Static Analysis".

When the results of the above described analysis seemed erroneous due to the exceedingly high pile capacities obtained, recent work by Berezantsev (14) and Vesic (13, 41, 42) was considered. Berezantsev, using a semi-empirical approach, developed values of Nq assuming local shear at the pile tip, rather than a fully developed failure surface. Vesic, from a series of tests on buried and driven piles developed values of Nq which compare fairly well with those of Berezantsev. A notable difference between the findings of Berezantsev and Vesic is that Berezantsev considers the effect of the zone of deformation at the pile tip only in calculating end bearing, while Vesic's work shows it to affect skin friction as well. Vesic found that the unit skin friction as well as the unit end bearing reach maximum values at critical depths ranging from 10 pile diameters to 30 pile diameters, depending on the density of the sand (13). He contends that..."at greater depths only punching shear occurs irrespective of the relative density of the sand. The unit point and skin resistances of the foundation increase linearly with depth only at shallow depths. At greater depths both resistances show a hyperbolic increase and reach asymptotically final values. These values are independent of overburden pressure and appear to be functions of the relative density of sand only."

For the analysis of end bearing, Berezantsev provides values of overburden reduction factors to reduce the effective overburden pressure. These factors are dependent upon the pile length, pile diameter, and angle of internal friction of the soil. Curves from which this factor is determined may be found in Figure 3. The overburden reduction factor reduces the overburden pressure in proportion to the depth below the ground surface and the angle of internal friction of the soil, thereby reducing the calculated value for the tip bearing capacity of piles. The skin friction is calculated in the same manner as in the Meyerhof analysis.

To utilize Vesic's findings in the analysis of skin friction, the unit overburden pressure is assumed to increase linearly from the surface to the "critical depth". At this point the unit overburden pressure becomes a constant due to, according to Vesic, the existence of a zone of decreased sand density adjacent to the pile at failure (13). The surrounding soils are thought to "arch" over the deformed sands leaving a relatively unstressed zone (13).

The critical depth may vary from 10 pile diameters to 30 pile diameters, depending on the density of the sand. The greater the density, the greater the critical depth. For this thesis, the sand density is firm to dense and a value of 25 pile diameters was selected for use as the critical depth.

Vesic also provides a table of values of Nq, determined from his tests on driven and buried piles, for use in analysis of end bearing. These values are lower than those presented by Berezantsev, and are based on a relatively small number of tests performed on circular model piles ranging in diameter from 2 to 6.75 inches.

Figure 4 presents curves of N_q vs. angle of internal friction from the work by Meyerhof (4), Berezantsev (14), and Vesic (41). Table 1 contains the soil parameters and data used in the static analyses.

Dynamic Analysis

Hiley Formula (3): The Hiley formula is thought to be one of the most accurate of the various dynamic formulas (1, 2, 39). It assumes that the ultimate capacity of the pile equals the final driving resistance, less energy losses within the system. It is as follows:

$$Ru = \frac{ef\ W_r\ h}{S + 1/2\ (C_1 + C_2 + C_3)}\ \frac{W_r + e^2 W_p}{W_r + W_p} \qquad (1)$$

Where:

Ru = Ultimate carrying capacity of pile

e_f = efficiency of hammer

W_r = weight of ram

h = height of fall of ram (for single-acting or drop hammers)

s = final set

Table 1. Static Analysis Data

RAYMOND AND HELCOR PILES
(Jetted And Then Driven a Few Feet)

Test Area	Zone From	To	ϕ (Degrees)	δ (Degrees)	$\tan\delta$	Ks	γ' (pcf)
Area 1	-8	-30	32	32	0.63	1	58
	-30	-33	--	--	400*	1	48
	-33	-48	34	34	0.68	1	58
	-48	-54	--	--	150*	1	48
	-54	-67	37	37	0.75	1	58
	-67	-75	32	32	0.60	1	63
Area 2	-8	-20	35	35	0.70	1	58
	-20	-41	38	38	0.78	1	58
	-41	-68	33	33	0.65	1	53
Area 3	-8	-43	34	34	0.68	1	63
PIPE PILES (Driven Mostly Without Jetting)							
Area 1	-8	-30	32	14	0.25	2	58
	-30	-33	--	--	400*	2	48
	-33	-48	34	17	0.30	2	58
	-48	-54	--	--	150*	2	48
	-54	-67	37	19	0.35	2	58
	-67	-75	32	14	0.25	2	63
Area 2	-8	-20	35	17	0.30	2	58
	-20	-41	38	19	0.35	2	58
	-41	-68	33	14	0.25	2	53
	-68	-72	41	22	0.40	2	63
Area 3	-8	-43	34	17	0.30	2	63

* Values of skin friction (in psf) for clay strata obtained from published tables (1).

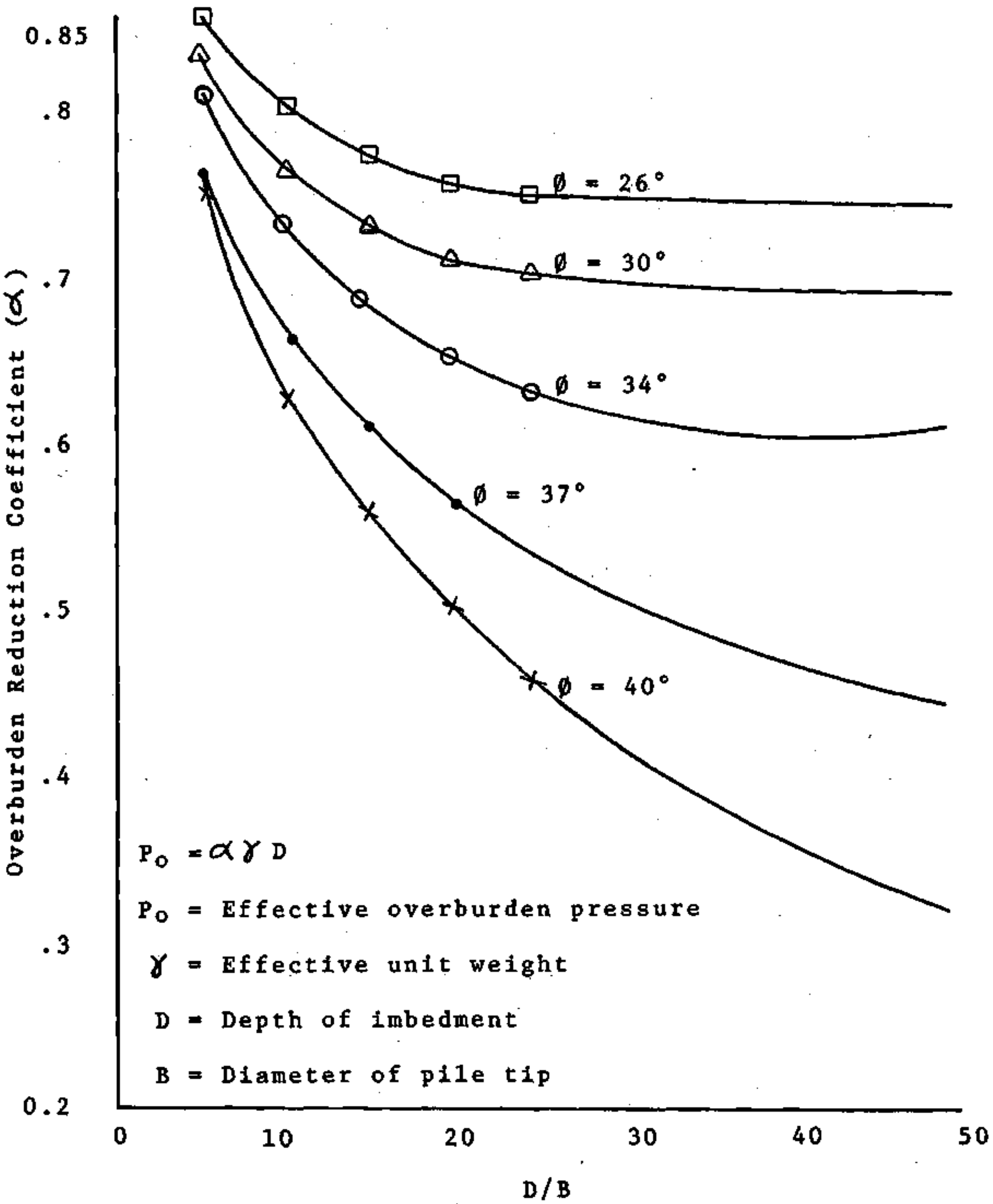


Figure 3. Overburden Reduction Factor vs. D/B For Varying Angles of Internal Friction

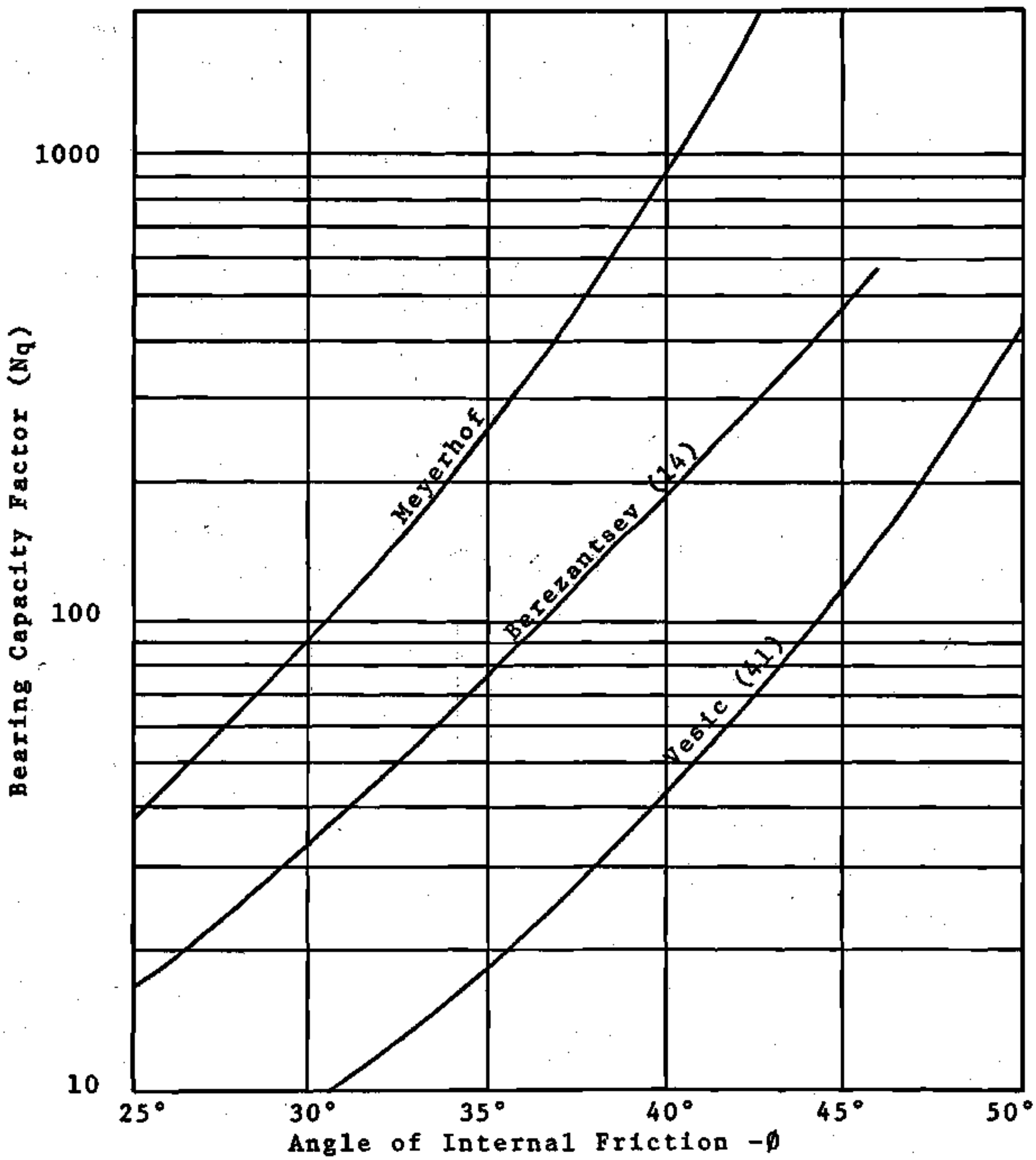


Figure 4.

Bearing Capacity Factors For Circular Deep Foundations

C_1 = temporary compression allowance for pile cap and head

C_2 = temporary compression of pile

C_3 = temporary compression allowance for ground

e = coefficient of restitution of cushion block

W_p = weight of pile

Worthy of special consideration is the second term of the right side of the equation. This term estimates the elastic rebound characteristics of the pile - capblock - hammer system. This term was derived using the analogy of two balls striking one another and thus does not account for pile length. Also worthy of mention is the assumption that the soil strength under dynamic loading (pile driving) is equal to the strength under static loading.

A complete derivation of the Hiley formula may be found in the original presentation by Hiley (36) or in other texts on pile driving -- as for example that of Chellis (1).

For use in the analysis, W_r, W_p, h, and e were furnished by the hammer, mandrel, and cushion block manufacturers, while ef and C_1 were obtained from published information (1). The values for S, C_2, and C_3 were measured in the field.

Engineering News Formula: The Engineering News formula is, perhaps, the most widely used dynamic formula in the United States. If, in the Hiley formula the impact loss is neglected, the hammer efficiency is taken to be 100 percent, the elastic energy losses in the cap, pile, and soil is represented by a constant term (1.0), h is taken in feet and multiplied by 12, and a factor of safety of 6 is assumed, the Engineering News formula results:

$$Ru = \frac{2\ W_r\ h}{S + 1.0} \qquad (2)$$

The formula was originally developed for use with drop hammers and wood piles in sand. However, it has been modified for use with single acting hammers by changing the term 1.0 to 0.1.

For use in the formula, W_r and h were furnished by the hammer manufacturer and S was measured in the field.

CHAPTER V

DISCUSSION OF RESULTS

The results of this study may be separated into two groups; those obtained directly from field tests, and those obtained from analysis of data obtained from the field tests. Pile driving and load testing data are contained in Table 1 and Table 2.

Results of Pile Load Tests

As previously stated, the purpose of the pile load tests was to determine the most suitable type of pile and to provide information as to the best method of installation of job piles. It accomplished these objectives.

The Hel Cor piles were found to provide more load carrying capacity than either the Raymond or pipe piles. The Raymond piles performed considerably better than did the pipes.

The Hel Cor piles not only exhibited less deflection under load than did the Raymond piles; they also drove shorter by several feet when driven to the driving resistance indicated by the Hiley formula. Notable examples are test piles 2-3 and 2-4, a Hel Cor pile and a Raymond pile driven side by side by identical procedures. Both piles were prejetted to elevation -35. The Hel Cor fell to elevation -36 under its own weight and reached the required driving criteria at elevation -41. The Raymond pile was also prejetted to elevation -35, but fell to elevation -38 under its own weight and reached the required driving criteria at elevation -50. It is noteworthy that both piles were taking up quickly when driving was stopped.

Pipe piles drove longer and developed less load carrying capacity than the other two pile types. With little or no aid from a prejet they drove deep at relatively low driving resistances and developed less load carrying capacity. For example, consider test pile 2-6, driven next to the two previously discussed piles. This pile was prejetted to elevation -35, fell to elevation -39 under its own weight, and reached the required driving criteria at elevation -69. This corresponds to a level 19 feet below the tip of the Raymond pile (T.P. 2-4) and 28 feet below the tip of the Hel Cor pile (T.P. 2-3). The performance of the Hel Cor and the pipe was comparable in that their total deflections were within 0.01 inches of each other (0.2584 vs 0.2684 for Hel Cor and pipe respectively). The Raymond pile failed under a load of 50 tons. The reason for the failure is not obvious. One may theorize that a small pocket of soft clay existed beneath the tip of the pile.

Of the 15 test piles, 7 failed at 2.5 times the design load (maximum load) or less. Four of the failures were pipe piles, two were Hel Cor, and one was Raymond. No explanation is available for the failure of the Raymond pile at a load equal to only half the intended maximum load. One of the Hel Cor failures (T.P. 1-3A) was a structural failure. A large tear in the shell prevented thorough cleaning, resulting in an obvious structural failure at a load of only 30 tons. The other Hel Cor failure was T.P. 1-4 which did not penetrate below the soft clay stratum known to exist beneath Test Area No. 1. This pile failed at a load of 70 tons.

Of the four premature pipe pile failures, only one can be attributed to anything other than low bearing capacity. Test pile 1-6 failed at the maximum load of 100 tons, but its tip was above the soft clay layer previously discussed. Test piles 2-6, 3-3, and 3-4 failed at loads of 80 to 90 tons with no explanation other than the driving criteria and the pile embedment was not sufficient to allow for the low skin friction developed by the smooth pile surface.

In order to drive the Hel Cor and Raymond piles to near their design elevation, extensive jetting was required, even to the point of using a side jet during driving. The pipe piles required little or no jetting to achieve their necessary penetration. If conditions were such that no minimum tip elevation criteria was needed, the superiority (greater carrying capacity with less penetration) of the corrugated Raymond and Hel Cor piles would probably be even more pronounced due to the greater skin friction developed by the corrugated piles.

Table 2. Summary of Pile Driving Data

Pile No.	Type	Tip Elevation	Blows For Last 6 Inches	Prejet Elevation	Side Jet Elev.	S(In.)	$C_2 + C_3$ (In.)	Calculated Capacity Tons (Hiley formula)
1-1	Raymond	-74.2	30	-44	-72	0.16	0.20	152.5
1-3A	Hel Cor 12-Inch	-72.4	25	-55	-55	0.18	0.32	110
1-4	Hel Cor 12-Inch	-46.0	22	-43	None	0.28	0.20	101
1-5	Pipe 12-Inch	-71.0	21	-35	None	0.22	0.47	145
1-6	Pipe	-45.0	14	-8	None	0.45	0.24	121
2-1	Raymond 12-Inch	-66.7	30	-50	None	0.12	0.20	172.5
2-2	Hel Cor 12-Inch	-58.0	28	-50	-50	0.19	0.22	123
2-3	Hel Cor	-41.1	38	-35		0.20	0.25	113
2-4	Raymond 12-Inch	-49.8	18	-35	None	0.36	0.10	102
2-5	Pipe 12-Inch	-69.3	37	-35	None	0.17	0.90	107
2-6	Pipe	-51.5	11	-8	None	0.61	0.54	78.5
3-1	Raymond 12-Inch	-42.5	23	-25	None	0.28	0.18	112
3-2	Hel Cor 12-Inch	-41.5	26	-35	None	0.20	0.28	110.5
3-3	Pipe 12-Inch	-43.0	14	-8	None	0.20	0.50	89.5
3-4	Pipe	-43.7	15	None	None	0.53	0.60	83.5

Table 3. Summary of Pile Load Test Data

Pile No.	Type	Failure Load (Tons)	Deflection (40 Tons) (In.)	Deflection (80 Tons) (In.)	Maximum Deflection (No Failure) (In.)	Final Deflection (No Failure) (In.)
1-1 (1)	Raymond	-	0.0804	0.2809	0.5628	0.3818
1-3A(2)	Hel Cor 12-Inch	30	-	-	-	-
1-4	Hel Cor 12-Inch	70	0.1793	-	-	-
1-5	Pipe 12-Inch	-	0.0753	0.2281	0.4356	0.1406
1-6	Pipe	90	0.1675	0.7376	-	-
2-1	Raymond 12-Inch	-	0.0710	0.1883	0.2891	0.1006
2-2	Hel Cor 12-Inch	-	(3)	0.2865	0.3995	0.1413
2-3	Hel Cor	-	0.0795	0.1812	0.2584	0.0906
2-4	Raymond 12-Inch	50	0.2259	-	-	-
2-5	Pipe 12-Inch	-	0.0626	0.1768	0.2684	0.0655
2-6	Pipe	90	0.0942	0.8423	-	-
3-1	Raymond 12-Inch	-	0.1150	0.2296	0.3537	0.1633
3-2	Hel Cor 12-Inch	-	0.0524	0.1216	0.1392	0.0640
3-3	Pipe 12-Inch	90	0.1440	0.8198	-	-
3-4	Pipe	90	0.2191	0.0716	-	-

(1) The pile may have been disturbed by jetting which took place during extraction of test pile 1-2A. At one time the jet was allowed to penetrate below the tip elevation of this pile.
(2) This pile suffered severe shell damage and appeared to fail structurally.
(3) The 40 Ton load increment was inadvertantly omitted.

In an attempt to obtain significant correlations, data from a load test program performed at the Jack Watson Steam Plant was studied. Data from this program may be found in the Appendix. At the Jack Watson Steam Plant, timber piles were driven by different procedures. A set of piles driven without jetting performed better under load than a set of jetted piles driven to the same criteria, although in some cases the jetted piles penetrated over twice as deep as the unjetted piles. For the procedures used and the data from the program, refer to Table 9 in the Appendix.

Results of Analyses

An attempt was made to correlate and explain the data gathered from the test pile program in terms of engineering theory. Initially the theoretical pile capacity was calculated using the Hiley formula, the Engineering News formula, and the previously described Meyerhof static analysis (2). As the results of this static analysis appeared unreasonably high, when compared to the actual pile performance and the results of the dynamic analyses, methods proposed by Berezantsev (14) and Vesic (13) were explored. The results of these analyses were compared with one-another as well as with the actual performance of the piles. The results of the comparison are shown in the form of a bar graph in Figure 5 and are listed in Table 5. This procedure is used for analysis discussed in detail in Chapter IV.

Table 4. Summary of Calculated Pile Capacities Using Static Methods of Analysis

Test Pile	Meyerhof Analysis			Berezantsev Analysis			Vesic Analysis		
	Skin Friction	End Bearing	Total	Skin Friction	End Bearing	Total	Skin Friction	End Bearing	Total
1-1 66' Raymond	144	124	268	144	37	181	100	10	110
1-3a 66' Hel Cor	143	244	387	143	74	216	98	23	121
1-4 38' Hel Cor	57	182	239	57	72	129	50	17	67
1-5 63' Pipe	135	272	407	135	82	217	93	31	124
1-6 37' Pipe	67	254	321	67	83	149	55	20	75
2-1 59' Raymond	145	131	276	145	45	190	101	25	126
2-2 50' Hel Cor	101	228	329	101	77	178	79	20	99
2-3 33' Hel Cor	66	515	581	66	134	200	57	16	73
2-4 42' Raymond	93	140	233	93	42	134	75	19	94
2-5 61' Pipe	155	470	625	155	262	417	98	82	200
2-6 44' Pipe	95	232	327	95	78	173	65	22	87
3-1 35' Raymond	65	143	208	65	46	111	51	20	71
3-2 34' Hel Cor	68	229	297	68	75	143	53	20	73
3-3 43' Pipe	58	263	321	58	86	144	46	20	66
3-4 44' Pipe	58	269	327	58	88	145	46	20	66

Table 5. Summary of Calculated Pile Capacities Static and Dynamic Analyses

Test Pile	Meyerhof (tons)	Berezantsev (tons)	Vesic (tons)	Engineering News* (tons)	Hiley (tons)	Failure Load (tons)
1-1	268	181	110	100	153	106
1-3A	387	216	121	98	110	30***
1-4	239	129	67	50	101	70
1-5	407	217	124	93	145	106**
1-6	321	149	75	55	121	90
2-1	276	190	126	101	173	--
2-2	329	178	99	79	123	--
2-3	581	200	73	57	113	--
2-4	233	134	94	75	102	50
2-5	625	417	200	98	107	--
2-6	327	173	87	65	79	90
3-1	208	111	71	51	112	112**
3-2	297	143	73	53	111	--
3-3	321	144	66	46	90	90
3-4	327	145	66	46	84	90

* Assumed safety factor = 6
** Estimated from load vs. settlement curve
*** Failed structurally

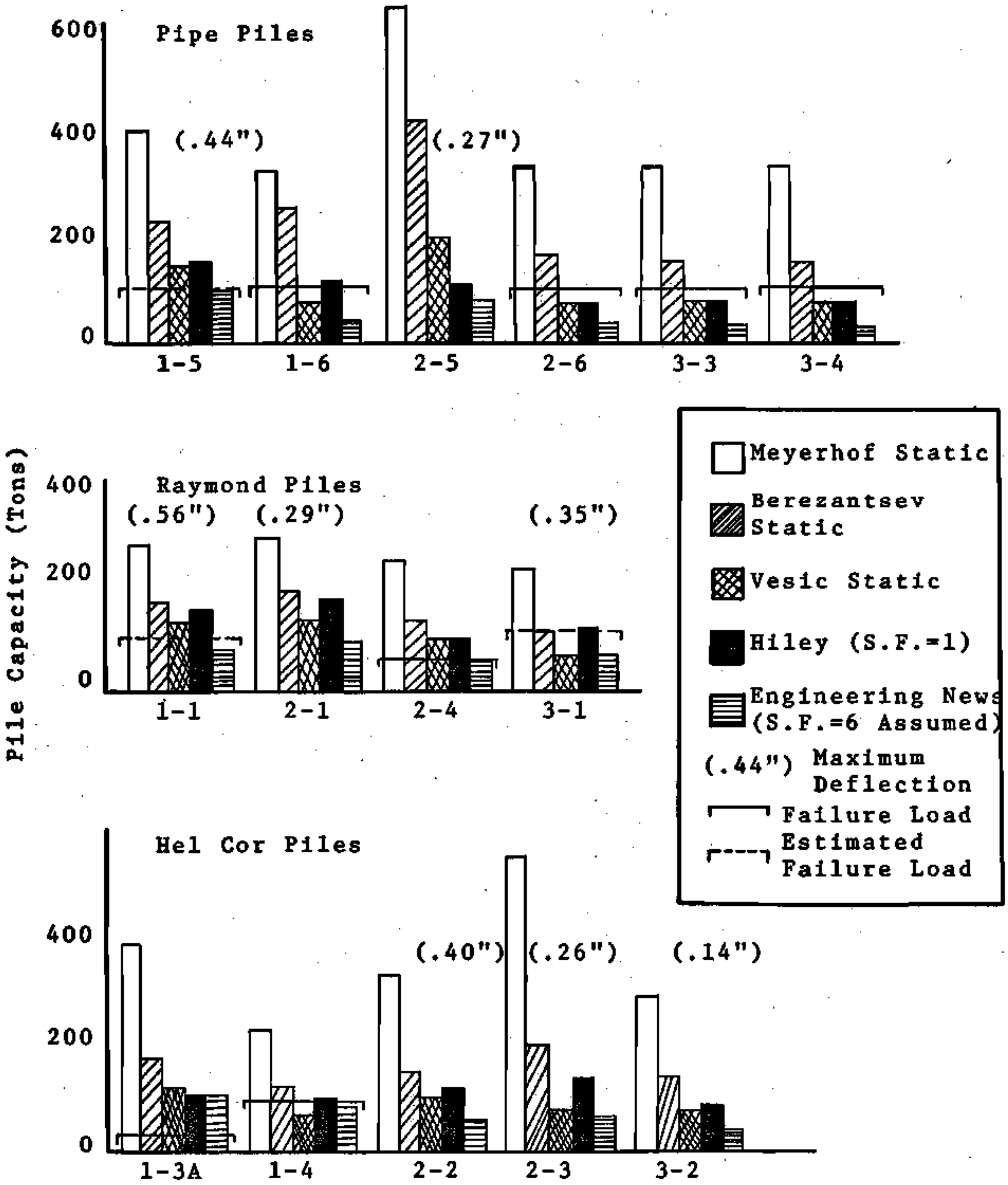


Figure 5.

Comparison of Pile Capacities

Table 6.

Relationship Between Hiley and Engineering News Formulas

Barry Steam Plant

Pile Type	Pile Number	Hiley Capacity (tons)	E. N. Capacity (tons)	Hiley/ E.N.	Hiley/ Actual	E.N./ Actual
Raymond	1-1	153	75	2.02	1.45*	.94*
	2-1	173	90	1.93	-	-
	2-4	102	43	2.40	2.04	.67
	3-1	112	52	2.16	1.00	.46
Hel Cor	1-3a	110	70	1.59	-	-
	1-4	101	52	1.96	1.44	.72
	2-2	123	68	1.82	-	-
	2-3	113	65	1.74	-	-
	3-2	111	65	1.70	-	-
Pipe	1-5	145	61	2.38	1.37	.88*
	1-6	121	36	3.40	1.35	.61
	2-5	107	72	1.49	-	-
	2-6	79	28	2.85	0.88	.72
	3-3	90	32	2.85	1.00	.51
	3-4	84	31	2.71	0.93	.51

* Failure load estimated by extending load vs. settlement curve.

Dynamic Analyses

When the capacities indicated by the two dynamic formulae were compared, it was found that the capacities indicated by the Hiley formula were higher than those indicated by the Engineering News formula. The bearing capacities determined by the Hiley formula were, in all but two cases (considering piles which were loaded to failure, or for which failure loads could be fairly accurately estimated), equal to or greater than the actual bearing capacity. For all piles which apparently did not approach failure, the Hiley formula indicated capacities in excess of 100 tons. Chellis (1) and Sowers (2) recommend the use of a safety factor of 2.5 with the Hiley formula. This safety factor would have been adequate for these piles, although the Hiley formula, in several cases, over estimated pile capacities by from 35 to 45 percent. It is interesting to note that the Hiley formula predicted the capacity of test piles 3-1 and 3-3 exactly. Test Pile 2-4 was not considered in the comparison because it failed at extremely low load (50 tons) without apparent reason.

The reason for the discrepancy between the Hiley formula capacity and the actual capacity for the majority of the piles may have been caused by the effect of jetting. At the Jack Watson Steam Plant, piles jetted and then driven a few feet had considerably less bearing capacity than piles driven without a jet, even though the unjetted piles were much shorter. Possibly, the detrimental effect on skin friction caused by jetting may not be reflected in the dynamic driving resistance.

The pile capacities indicated by the Engineering News formula are nearly as close to the actual pile capacities as the capacities indicated by the Hiley formula. The values determined from the Engineering News formula are in all cases on the safe side. Ratios of Engineering News safe capacity to actual capacity range from 0.46 to 0.94, indicating actual safety factors ranging from about one to slightly more than two. It should be noted that the Engineering News formula supposedly contains a theoretical safety factor of six.

For the relationships between the Hiley formula and Engineering News formula, and actual pile capacity, refer to Table 5.

It has been stated by Chellis (1) that a theoretical safety factor of six is included in the Engineering News formula. However, Sowers (2) states that the actual safety factor may vary from less than 1 to as much as 30. According to Flaate (39), "The Engineering News formula should not be used, as better formulas are available."

However, data from this program indicates that the use of the Engineering News formula, in the case of piles jetted and then driven a few feet, may be advisable. The data from this program indicates that as an average, the Hiley formula

(S.F.=1) yielded a theoretical pile capacity 2.2 times that yielded by the Engineering News formula. The maximum ratio of the two was 3.4 while the lowest was 1.5 The Hiley formula capacities were, in most cases, larger than the actual bearing capacity of the piles by from 35 to 45 percent.

When the different types of piles were examined separately, the following relationships were obtained:

Pile Type	Hiley/E.N.	Maximum Variation From Mean
Hel Cor	1.76	11.4%
Raymond	2.13	12.6%
Pipe	2.61	43.0%

The relationships for individual piles are contained in Table 5.

Since the Engineering News formula was originally developed for use with timber piles, the results of the Jack Watson Steam Plant program were correlated. There was a definite relationship between the two formulas. The mean pile capacity indicated by the Hiley formula (S.F.=1) was 1.73 times that indicated by the Engineering News formula. The maximum variation from the mean was 17.3%. However, six of the nine piles studied had a maximum variation from the mean of only 0.3%. It is interesting to note that the piles which were studied included piles driven without a jet and piles jetted and then driven.

The results of this comparison compare very well with the Hel Cor piles at Barry Steam Plant and compare fairly well with the Raymond Piles. However, the ratio between the two formulas for the pipe piles was quite high. This could be due to the high elasticity of the pipe piles as compared to the timber piles or to the Raymond or Hel Cor piles.

Static Analyses

Initially the theoretical pile capacity was determined by the Meyerhof analyses. The calculated capacities were quite high, as shown in Figure 5; being in almost all cases much greater than the observed values and those indicated by other analyses. These high values appeared unreasonable and were of course, on the unsafe side. The reason for the high calculated capacities was found to be an extremely high value of tip bearing, due to the use of Meyerhof's values for N_q. These values assume a fully developed failure zone as well as an increase in sand density near the pile shaft due to pile driving.

The pile capacities determined by the Meyerhof method exceeded the actual failure load of the piles by from two to five times. The largest discrepancies were encountered in cases where the pile tip was within a zone of sand with a high angle of internal friction (Test Piles 2-3 and 2-5). The Meyerhof bearing capacity factors increase tremendously with increases in angle of internal friction, resulting in excessively high calculated values of end bearing. The individual pile capacities may be found in Table 4 and Table 5.

The capacities were then calculated using the method of analysis developed by Berezantsev (14). The capacities calculated using this procedure were often more than 50 percent lower than those indicated by the Meyerhof analysis. The Berezantsev values, however, were still higher than the actual bearing capacity of the piles in all but one case (Test Pile 3-1). The ratio between the calculated and actual value of bearing capacity varied from 60 percent (Test Pile 3-3) to 170 percent (Test Pile 2-4) of the actual bearing capacity. The actual calculated capacities may be found in Table 4 and Table 5.

These discrepancies may be explained by the use of the same method of skin friction analysis as was used in the Meyerhof analysis. Work by Vesic (13, 41, 42) has cast considerable doubt on this method of analysis. Further, the effect of jetting on the corrugated piles certainly would decrease the effective skin friction to below that indicated by the Meyerhof analysis.

Finally, the method of analysis proposed by Vesic was used. The capacities obtained from this method of analysis were substantially lower than those obtained from the Berezantsev analysis. The difference between the two methods varied from 30 percent (Test Pile 2-4) to 68 percent (Test Pile 2-3). However, all but these two piles fell in the range of 35 to 55 percent difference, with the results of the Vesic analysis being, in all cases, the lower of the two.

The lower results of the Vesic analysis can easily be explained by considering the lower value of skin friction obtained, and the lower values of N_q presented by Vesic (41) for use in the analysis of end bearing.

The results of this analysis compare more closely with the actual pile capacities than do the results of the other methods of analysis, both static and dynamic.

The Vesic analysis indicated pile capacities above the actual or estimated failure load in two of eight cases. The calculated values for Test Piles 1-1 and 1-5 exceeded the failure loads (which were estimated from the load vs. settlement curves) by 4 and 14 percent, respectively. The remainder of the calculated capacities were less than the failure loads by amounts varying from 3 percent to 58 percent. The overall average descrepancy between calculated and actual capacity was -11.7 percent. In drawing a comparison, the failure loads of Test Piles 1-3A and 2-4 were not considered. Test Pile 1-3A failed structurally and Test Pile 2-4 failed at a very low load for an unknown reason.

The Vesic analysis is the only static method of analysis that indicated capacities which were close to the actual capacities or in error on the conservative side. This may be, to some extent, due to Vesic's theory of a zone of reduced stress around the pile shaft being particularly compatible with jetted piles. Also, his bearing capacity factors (N_q) were empirically determined and are substantially lower than are those of Berezantsev. The relationship between the values of N_q as presented by Meyerhof, Berezantsev, and Vesic may be seen in Figure 4.

Fallacies Encountered

Among the significant results of this investigation are certain fallacies which were found to exist in many of the assumptions and procedures used to analyze pile performance. In addition to the problems involved in selecting the method with which to analyze the piles, numerous specific discrepancies between calculated and measured values were noted, particularly within the Hiley formula.

Of significance are fallacies noted within the Hiley formula, which is accepted as being one of the most accurate if not the most accurate of the dynamic formulas (1, 2, 39). Test pile 2-5, a pipe pile driven almost to refusal, revealed a glaring fault. When driving was stopped, this pile had a very low set (0.17 inches) and a very high elastic rebound (0.90 inches). Since both the set and elastic rebound of the pile appear in the denominator of the Hiley formula, the high elastic rebound served to decrease the calculated pile capacity to nearly the same as that of test piles 1-6, 2-6, 3-3 and 3-4, all of which had much higher final sets, and all of which failed at less than the indicated maximum capacity. The Hiley formula appears, therefore, to be somewhat erratic and less reliable in the case of a long elastic pile, in that it underestimated the capacity of a pile driven almost to refusal. Also it somewhat overestimated the capacity of four piles driven only to the indicated driving resistance.

Another fallacy is found in the commonly used method of field determination of the elastic rebound of the pile ($C_2 + C_3$). According to Hiley (37) C_2 is the elastic compression of the pile under driving stresses. The commonly used method of determining ($C_2 + C_3$) is by measurement near the point at which the pile enters the ground. In the case of a long pile driven only a short distance, a substantial portion of the elastic deflection may take place above the point of measurement. Failure to account for this could lead to substantial overestimation of the pile capacity.

Chellis (1) recognizes the problem, but does not offer a solution. Calculations with hypothetical pile capacities taking into account the elastic compression above the measurement point, indicate capacities of about 80 to 90 percent of those indicated using values of elastic compression determined near the point at which the pile enters the ground. A sample comparison is as follows:

$$Ru = \frac{(\text{Constant} = K)}{S + 1/2\ (C_1 + C_2 + C_3)}$$

Assume: $S = 0.2''$

$(C_2 + C_3)m = 0.2''$

$C_1 = 0.1$

$$\therefore \quad Ru = \frac{K}{0.2 + 1/2\ (0.1 + 0.2)}$$

$$\text{or} \quad Ru = \frac{K}{0.35} = 2.9\ K$$

Now consider the same pile driven to the same driving resistance at only one half the depth. Therefore, one half of the pile length will be above the measurement point. As C_2 is a function of pile length, it will be doubled: Assuming $C_3 = 0.1$

$$Ru_o = \frac{K}{0.2 + 1/2\ (0.1 + 0.2 + 0.1)} = \frac{K}{0.4}$$

$$\therefore \quad Ru_o = 2.5\ K$$

$\frac{Ru}{Ru_o} = 1.2$ or 20% too high

Furthermore, the presently used methods for estimating C_2, the elastic compression of the pile, do not appear accurate. Published tables (1) indicate that C_2 varies with the average driving stress throughout the pile. The driving stress is related to a factor which is multiplied by the length to the center of driving resistance of the pile. Using this factor and published tables (1), C_2 may be estimated. To investigate the accuracy of the empirical factors, the driving stresses were determined for each pile by dividing the pile capacity (determined from the formula and measured values of set and elastic rebound) by the cross-sectional area of the pile. C_2 was then determined using published constants (1) and an assumed value for the length to the center of driving resistance. The actual C_2 was determined by subtracting (C_3=0.1) from the field measurement of the elastic rebound. The comparison of the two values showed variations of actual C_2 of as much as 397 percent above empirical C_2 and 480 percent below empirical C_2. Errors of this magnitude could not have been caused by an incorrect assumption of the length to center of driving resistance of the pile . Data and comparison of measured and estimated values may be found in Table 5.

Although the errors in the estimation of elastic rebound are very large, the resultant error in calculated pile capacity is fairly small -- approximately 10 to 20 percent. Errors of this magniture are however, significant. To avoid this error, the actual field measurements should always be substituted back into the equation to verify the pile capacity.

Table 7.

Comparison of Measured to Calculated Values of C_2 and C_3

Test Pile	Type	Tip Elev.	Measured Set (in.) S_m	Measured Bounce $(C_2+C_3)_m$	C_2 Measured(*) C_{2m}	Total Pile Length	Assumed L (L_o)	Calculated C_2 C_{2c}	Adjusted L (L_a)	L_a/L_o
1-1	R	-74.2	0.16	0.20	0.10	84'	70'	0.082	85.5'	1.2
1-3a	H	-72.4	0.18	0.32	0.22	80'	70'	0.06	257'	3.7
1-4	H	-46.0	0.28	0.20	0.10	60'	55'	0.035	157'	2.9
1-5	P	-71.0	0.22	0.47	0.37	75'	75'	1.19	31'	0.3
1-6	P	-45.0	0.45	0.24	0.14	60'	55'	0.68	12'	0.2
2-1	R	-66.7	0.12	0.20	0.10	84'	70'	0.09	78'	1.1
2-2	H	-58.0	0.19	0.22	0.12	80'	70'	0.062	135'	1.9
2-3	H	-41.1	0.20	0.25	0.25	60'	55'	0.044	341'	3.4
2-4	R	-49.8	0.36	0.10	0.01**	56'	45'	0.036	13'	0.3
2-5	P	-69.3	0.17	0.90	0.80	75'	75'	0.90	67'	0.9
2-6	P	-51.5	0.61	0.54	0.44	60'	55'	0.47	52'	0.9
3-1	R	-42.5	0.28	0.18	0.08	56'	45'	0.039	92'	2.1
3-2	H	-41.5	0.20	0.28	0.18	60'	55'	0.044	225'	4.1
3-3	P	-43.0	0.52	0.50	0.40	60'	55'	0.54	40'	0.7
3-4	P	-43.7	0.53	0.60	0.50	60'	55'	0.45	61'	1.1

* Assuming C_3 = 0.1

** Said to be 0.01 for purposes of calculation

CHAPTER VI

CONCLUSIONS

The conclusions drawn from this analysis are as follows:

1. The Hel Cor pile drives shorter and out performs the Raymond step taper pile when the piles are jetted and then driven into firm to dense sands, if the piles are driven to equivalent dynamic driving resistances. Pipe piles driven to a resistance equivalent to the same capacity, drive deeper and develop less real bearing capacity. This is attributable to the smaller skin friction developed by the smooth pile surface.
2. Jetting, although allowing greater penetration, may serve to decrease pile capacity to below that for a shorter, unjetted pile and may reduce the reliability of both dynamic and static analyses.
3. In sedimentary deposits, a very extensive subsurface investigation is necessary to locate deposits of soft material which could cause localized pile failures.
4. The results of the Hiley formula analysis were more consistent than those of the Engineering News formula

when compared to the actual load carrying capacity of the piles. The commonly recommended safety factor of 2.5 would have been adequate for the piles that were analyzed.

5. The actual safety factors determined from the Engineering News formula varied from one to two, even though the Engineering News formula supposedly contains a safety factor of six. The use of the Engineering News formula to predict the capacity of the piles analyzed by this thesis would have resulted in dangerously low design capacities.

6. Correlations between the results of the Hiley formula and Engineering News formula for the Raymond, Hel Cor, and pipe piles at Barry Steam Plant are somewhat erratic. There is, however, a well defined relationship between the capacities calculated by these two formulas for timber piles driven at the Jack Watson Steam Plant.

7. At Barry Steam Plant, there is a relationship between driving resistance and pile capacity, although not as well defined as at Jack Watson Steam Plant.

8. The bearing capacity factors developed by Meyerhof are unreasonably high. They were entirely unrealistic and thus cannot be used for a reliable prediction of pile capacity.

9. The bearing capacity factors developed by Berezantsev are more conservative than those of Meyerhof. However, for the piles analyzed for this thesis, these values also appear to be too high.

10. The bearing capacity factors developed by Vesic are the lowest of the factors considered by this thesis. For the piles analyzed, they appear to be the most nearly correct, and are in most cases, conservative.

11. The use of the method of skin friction analysis recommended by Meyerhof and Berezantsev is not realistic, particularly in the case of jetted piles. Vesic's method should be used to avoid calculated pile capacities seriously in error on the unsafe side.

Perhaps the most noteworthy results of this investigation are the fallacies encountered within commonly used methods of analysis. These include:

1. In cases where highly stressed pile, which deforms greatly when driven, is driven nearly to refusal, the Hiley formula indicates an unreasonably low capacity due to the high elastic rebound.

2. When making a field determination of the elastic rebound of a pile during driving, the portion of the pile above the point of measurement should not be neglected, as is common practice.

3. The Chellis constants (C_1 and C_2) for estimating the elastic rebound of a pile are grossly in error.

This study points out, by the lack of correlations and by the fallacies revealed, the need for continued study of results of full scale pile load tests. The writer is of the opinion that the value of controlled laboratory research is small compared with the value of study of field data.

The importance of the work by Berezantsev and Vesic, particularly that of Vesic, is great. However, much more work is needed since the relationships established by Vesic are based on relatively few tests and are confined to use with uniform consistency, fairly clean sand. Additional research to expand this work could include:

1. The effect of ground water.
2. The effect of corrugations and pile taper.
3. The effect of particle size and silt content.
4. The behavior of piles in a stratified deposit.

To investigate the above factors will require extensive research programs involving high costs. Rather than wait for this type of program, the independent researcher or thesis author should use the newly presented methods of analysis to analyze other test pile programs similar to the one analyzed by this thesis. This additional work will help to verify the new methods of analysis and thereby speed their acceptance by the Civil Engineering profession.

APPENDIX

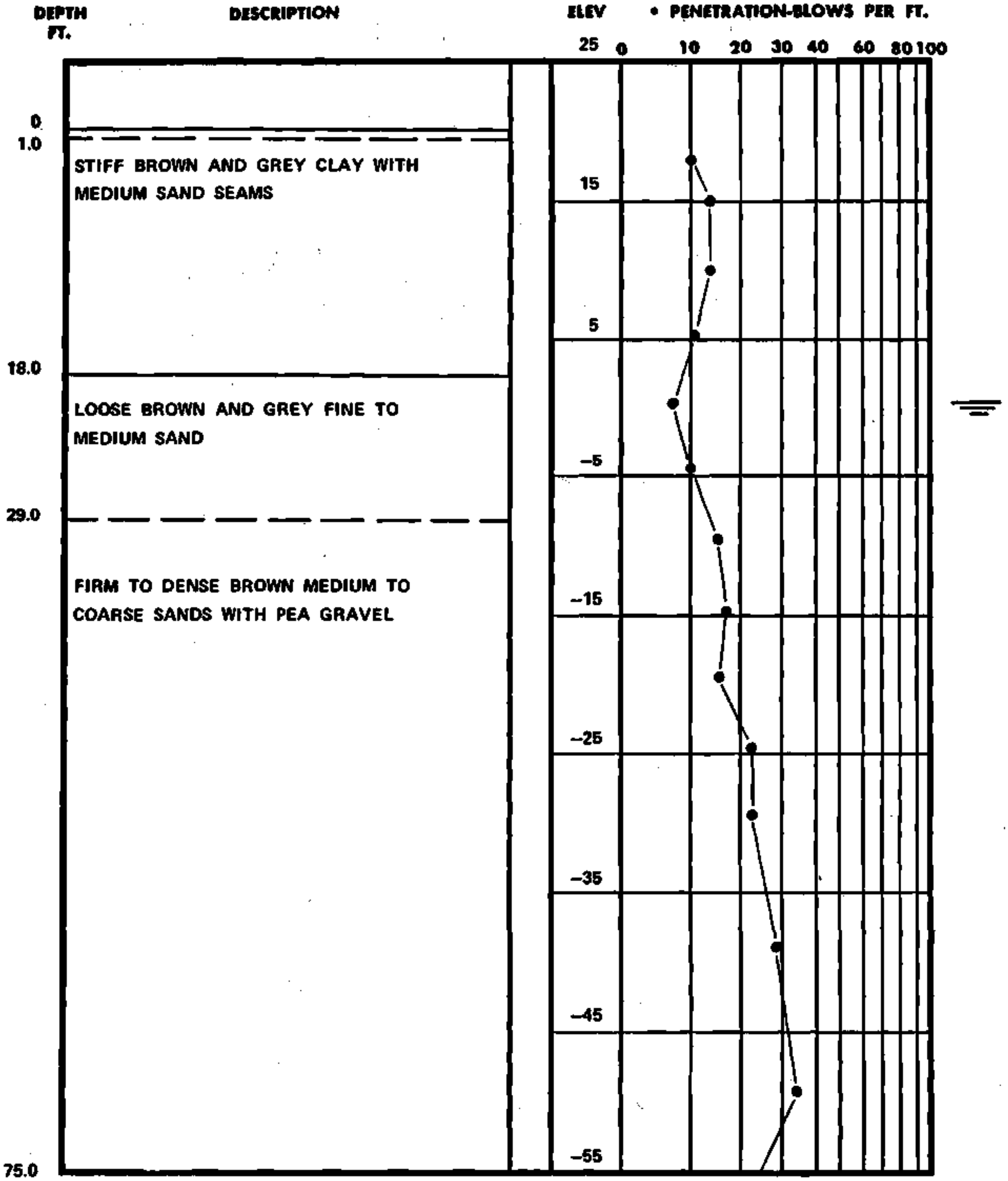


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Figure 6. Test Boring Record
Boring Number B-108

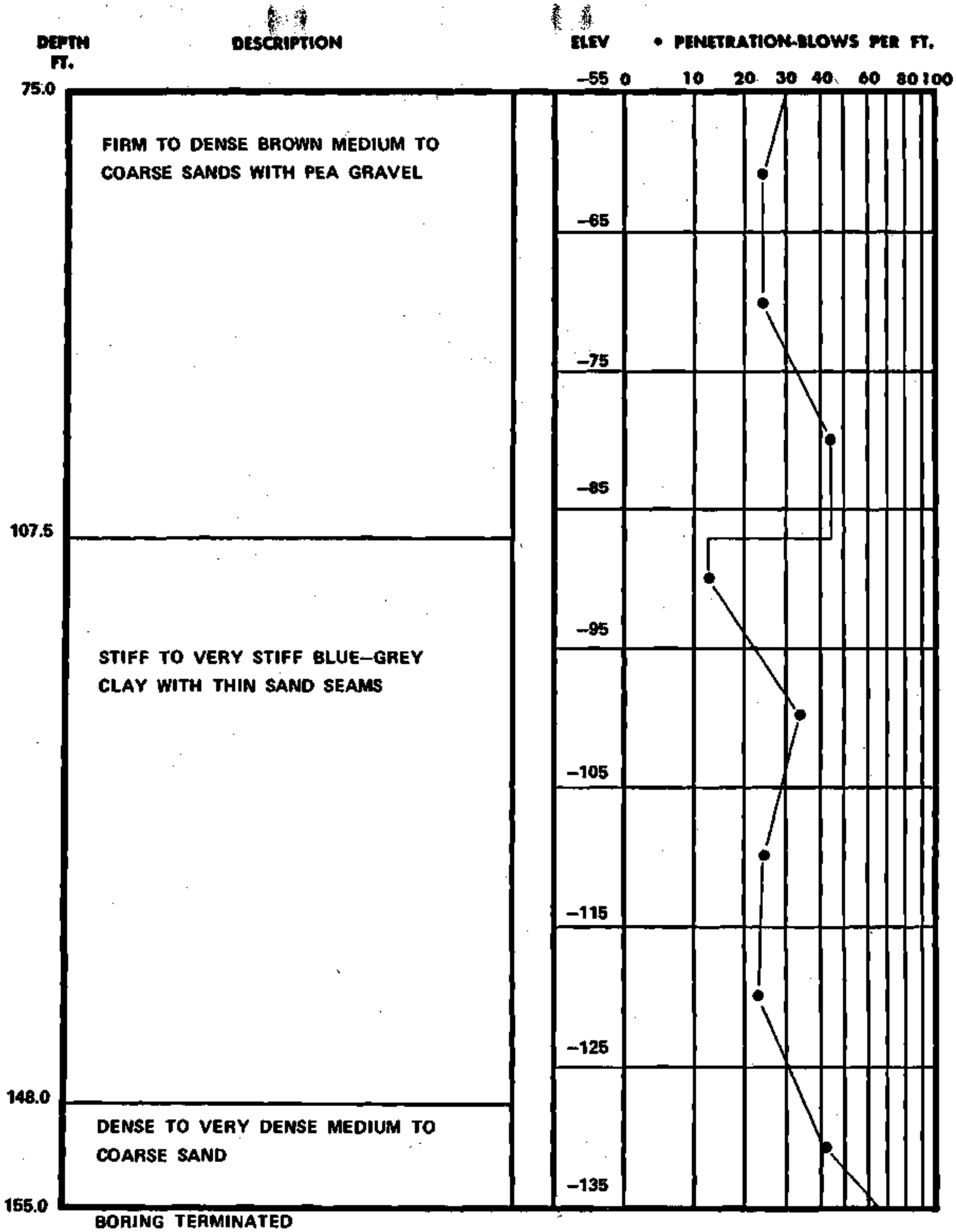

Figure 6. (continued)

Test Boring Record
Boring Number B-108

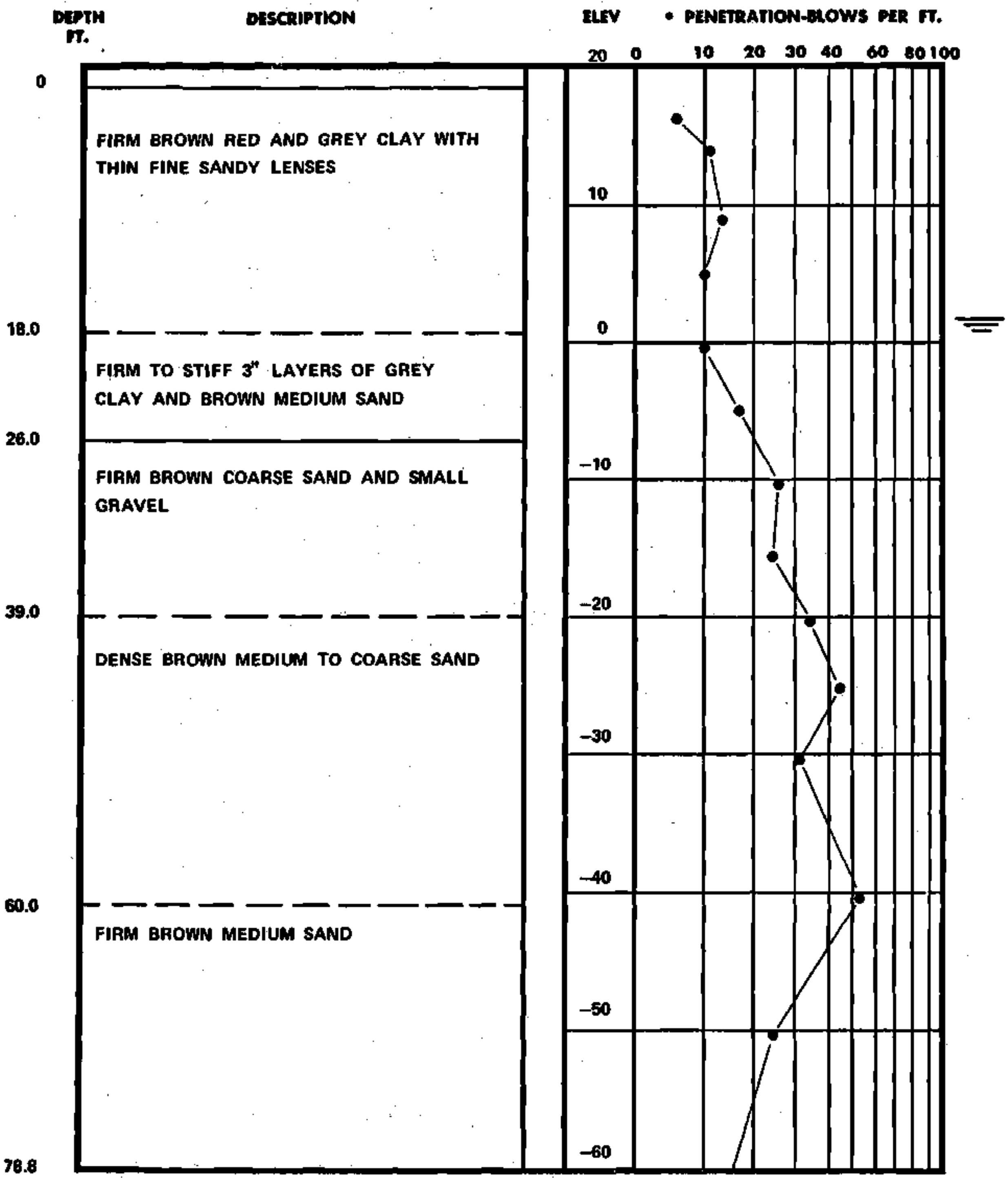

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Figure 7. Test Boring Record
Boring Number B-109

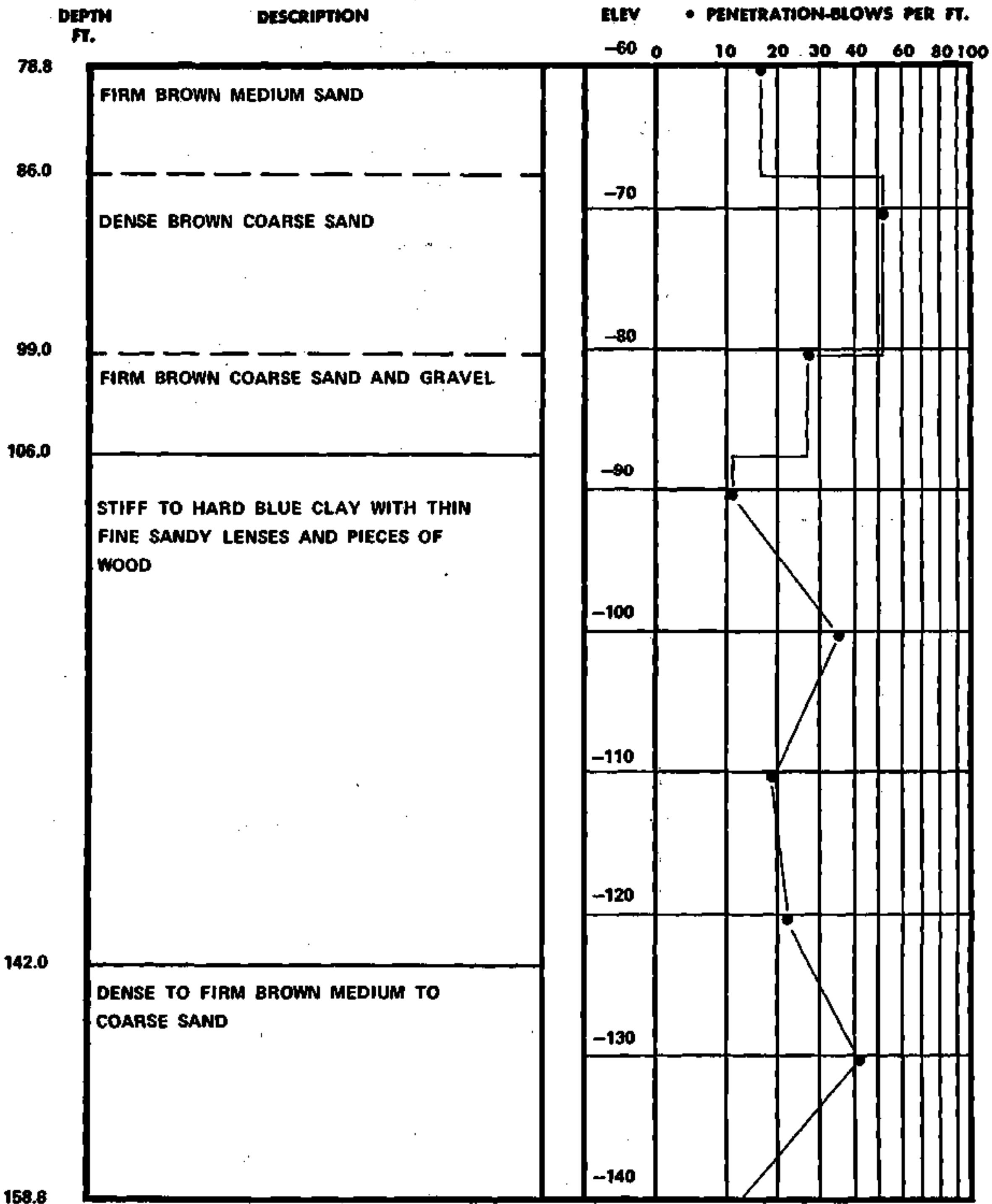

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Figure 7. (continued)

Test Boring Record
Boring Number B-109

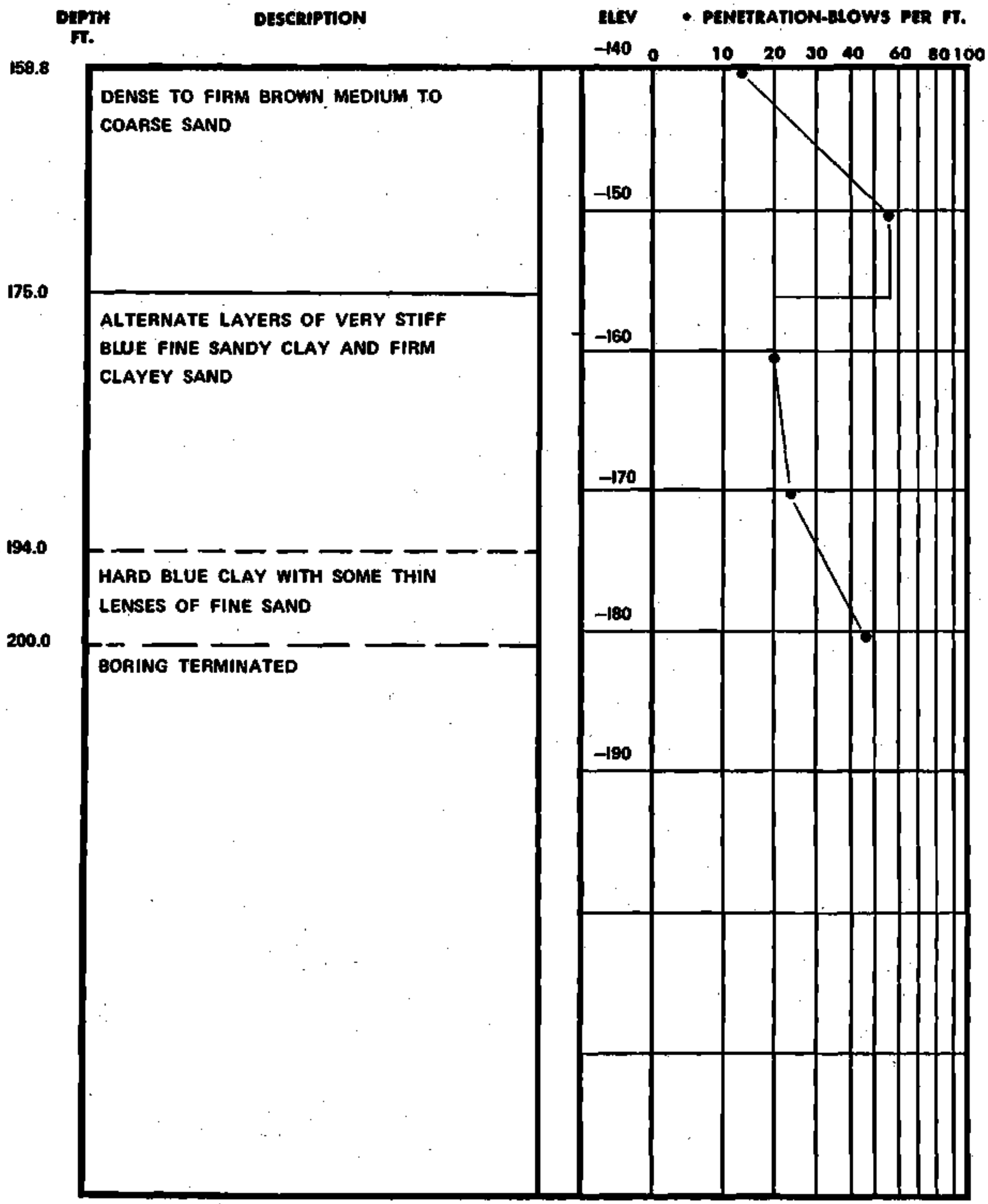

Figure 7. (continued)

Test Boring Record
Boring Number B-109

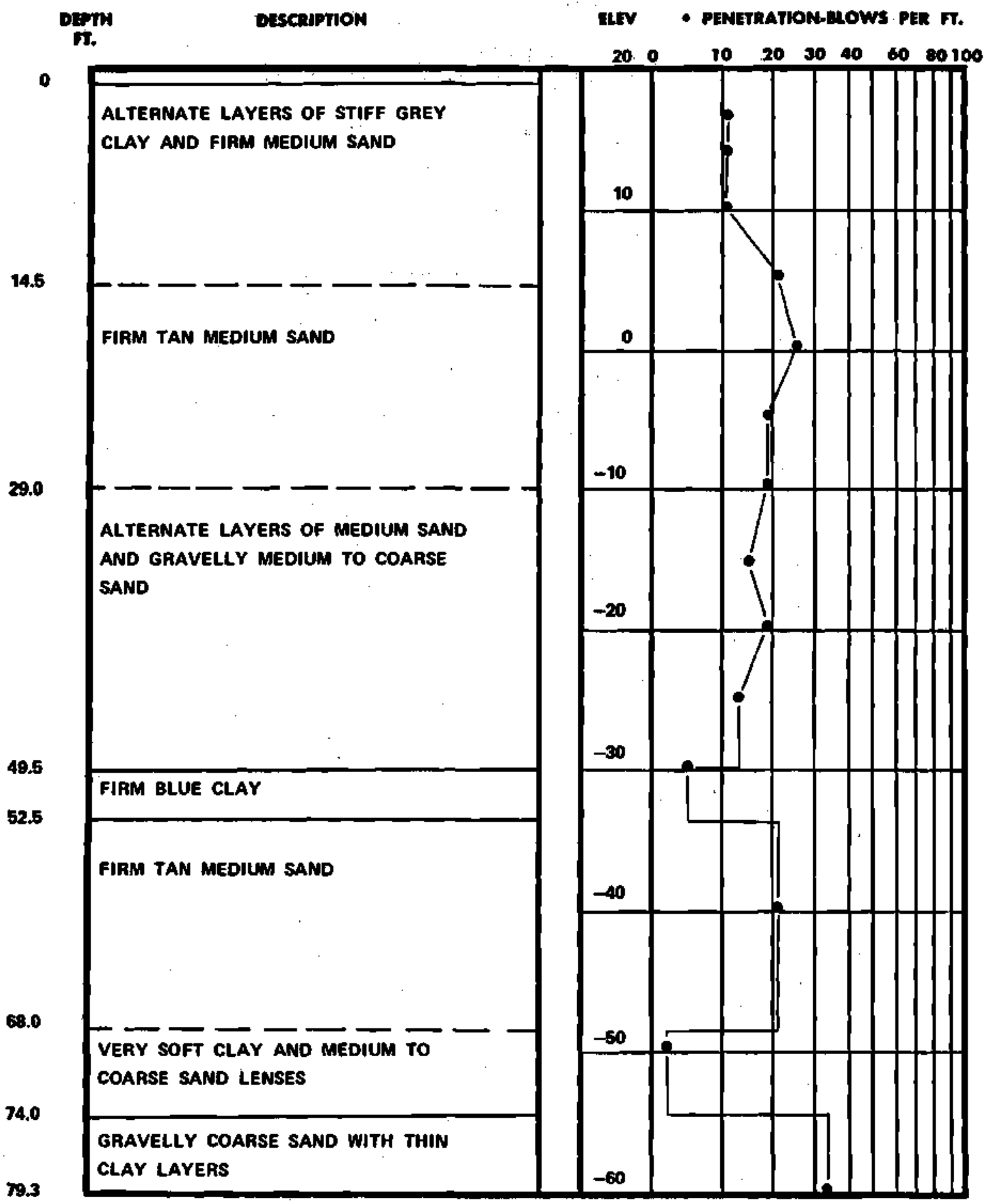

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Figure 8. Test Boring Record
Boring Number B-112

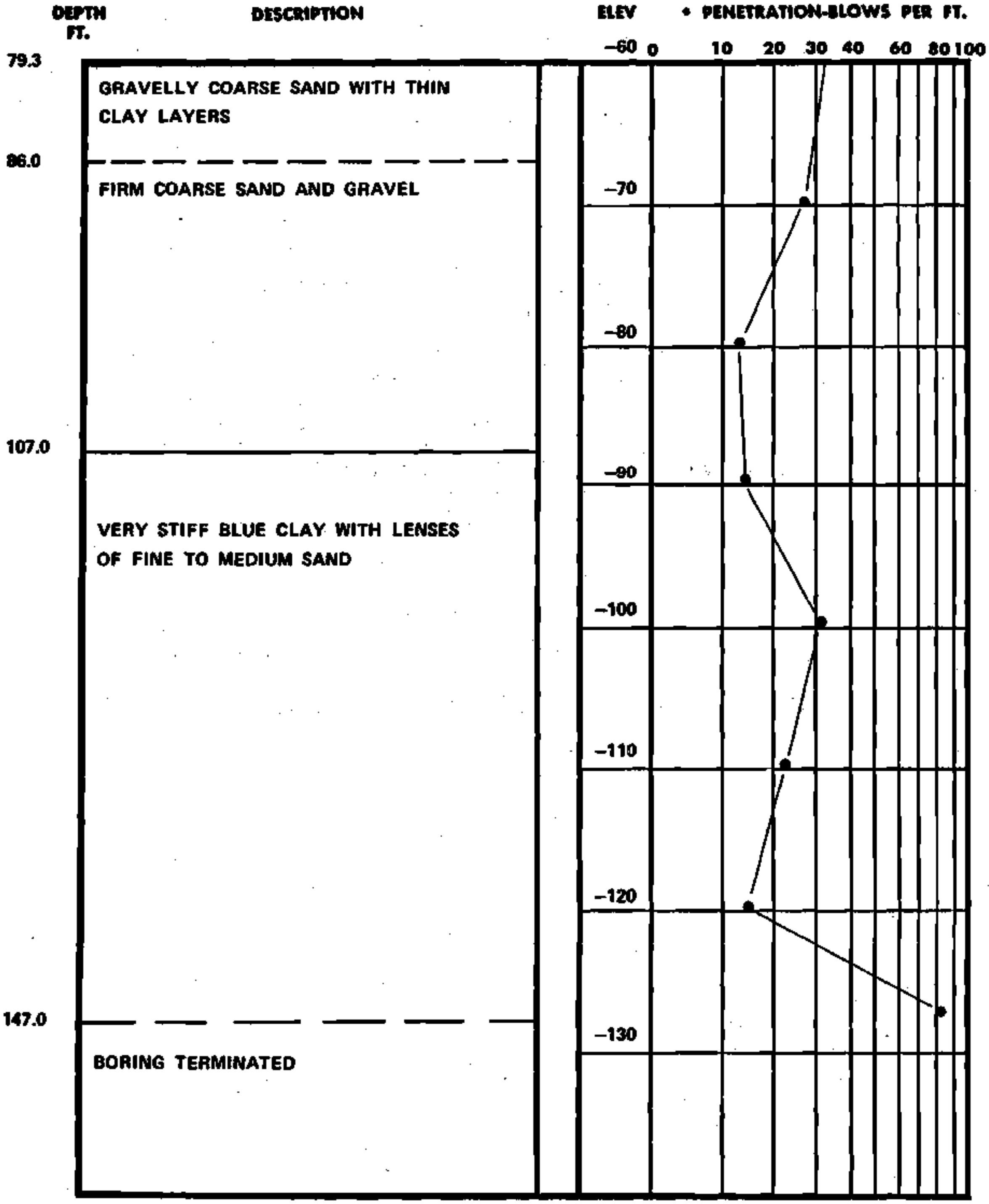

Figure 8. (continued)

Test Boring Record
Boring Number B-112

Table 8.

Record of Pile Driving

Pile Number: 1-1
Pile Type: Raymond
Original Pile Length: 84'

Pile Tip Elev.	Blows Per Foot
*	*
31	3
32	4
33	4
34	-
35	-
36	-
37	11
38	10
39	11
40	11
41	11
42	12
43	21
44	30
45	35
46	40
47	40
48	33
49	30
50	25
51	20
52	24
53	20
54	20
55	20
56	12
57	14
58	18
59	20
60	34
61	30
62	30
63	30
64	29
65	26
66	33
67	34
68	40
69	39
70	46
71	40
72	46
73	58
74	58
75	36/6"

* Prejetted to elevation -44
Side jetted at elevation -45 until tip reached elevation -72.

Table 8.(continued)

Record of Pile Driving

Pile Number: 1-3A
Pile Type: 12" Hel Cor
Original Pile Length: 80'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	61	2
51	3	62	4
52	3	63	5
53	5	64	10
54	10	65	14
55	15	66	13
56	29	67	33
57	--	68	90/9"
58	30	69	(50 for first
59	20	70	6", 40 for
60	4	71	last 3")

* Prejetted to elevation -55
Side jetted at elevation -55 until tip reached elevation -67.

Pile Number: 1-4
Pile Type: 12" Hel Cor
Original Pile Length: 60'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	43	35
39	5	44	39
40	10	45	45
41	15	46	48
42	27		

* Prejetted to elevation -43
No side jet.

Table 8. (continued)

Record of Pile Driving

Pile Number: 1-5
Pile Type: 12" Pipe
Original Pile Length: 80'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	56	18
40	7	57	26
41	10	58	25
42	12	59	25
43	14	60	24
44	16	61	25
45	22	62	25
46	17	63	24
47	18	64	24
48	22	65	24
49	24	66	24
50	23	67	25
51	23	68	27
52	19	69	24
53	13	70	22
54	14	71	20
55	17	72	36

* Prejetted to elevation -35
No side jet.

Table 8. (continued)

Record of Pile Driving

Pile Number: 1-6
Pile Type: 12" Pipe
Original Pile Length: 60'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	32	13
19	7	33	13
20	7	34	13
21	9	35	15
22	9	36	15
23	11	37	16
24	13	38	17
25	11	39	10
26	13	40	18
27	12	41	18
28	14	42	20
29	15	43	11
30	14	44	21
31	13	45	20
		46	23

* Prejetted to elevation -8
No side jet.

Pile Number: 2-1
Pile Type: Raymond
Original Pile Length: 84'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	63	8
58	2	64	10
59	2	65	10
60	2	66	20
61	5	67	32
62	6	68	50
		69	(30 blows for last 6")

* Prejetted to elevation -50
No side jet.

Table 8. (continued)

Record of Pile Driving

Pile Number: 2-2
Pile Type: 12" Hel Cor
Original Pile Length: 80'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	56	--
53	12	57	49
54	22	58	53 (28 blows for last 6 inches)
55	37		

* Prejetted to elevation -50
Side jetted at elevation -55 until pile tip reached elevation -57.

Pile Number: 2-3
Pile Type: 12" Hel Cor
Original Pile Length: 60'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	41	52/10"
39	21	42	38 for last 6 inches (tip elev. = 41.5)
40	37		

* Prejetted to elevation -35
No side jet.

Table 8. (continued)

Record of Pile Driving

Pile Number: 2-4
Pile Type: Raymond
Original Pile Length: 56'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	45	11
38	11	46	13
39	8	47	15
40	8	48	22
41	8	49	30
42	5	50	30
43	8	51	33
44	11		(18 blows for last 6")

* Prejetted to elevation -35
No side jet.

Pile Number: 2-5
Pile Type: 12" Pipe
Original Pile Length: 80'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	56	16
40	8	57	13
41	10	58	12
42	9	59	14
43	-	60	14
44	-	61	14
45	12	62	16
46	13	63	20
47	14	64	21
48	16	65	20
49	17	66	20
50	16	67	22
51	18	68	30
52	17	69	37
53	19	70	56
54	18	71	37/6" @
55	18		70.5 elev.

* Prejetted to elevation -35
No side jet.

Table 8. (continued)

Record of Pile Driving

Pile Number: 2-6
Pile Type: 12" Pipe
Original Pile Length: 60'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	32	9
13	5	33	9
14	3	34	9
15	2	35	9
16	2	36	10
17	3	37	13
18	7	38	13
19	7	39	14
20	6	40	13
21	9	41	14
22	8	42	16
23	8	43	15
24	8	44	15
25	8	45	16
26	9	46	16
27	10	47	18
28	10	48	20
29	9	49	18
30	10	50	19
31	10	51	19
		52	11/6"

* Prejetted to elevation -8
No side jet.

Table 8. (continued)

Record of Pile Driving

Pile Number: 3-1
Pile Type: Raymond
Original Pile Length: 54'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	37	14
31	5	38	17
32	7	39	22
33	10	40	26
34	11	41	31
35	12	42	35
36	13	43	23/6"

* Prejetted to elevation -25
No side jet.

Pile Number: 3-2
Pile Type: 12" Hel Cor
Original Pile Length: 60'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	40	26
36	5	41	34
37	9	42	48
38	14		(26 for
39	16		last 6")

* Prejetted to elevation -30
No side jet.

Table 8. (continued)

Record of Pile Driving

Pile Number: 3-3
Pile Type: 12" Pipe
Original Pile Length: 60'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	29	9
14	3	30	9
15	3	31	11
16	5	32	13
17	4	33	12
18	4	34	13
19	6	35	12
20	6	36	14
21	5	37	18
22	6	38	21
23	7	39	22
24	6	40	24
25	8	41	24
26	9	42	24
27	10	43	23
28	11	44	14/6"

* Prejetted to elevation -8
No side jet.

Table 8. (continued)

Record of Pile Driving

Pile Number: 3-4
Pile Type: 12" Pipe
Original Pile Length: 60'

Pile Tip Elev.	Blows Per Foot	Pile Tip Elev.	Blows Per Foot
*	*	26	9
7	3	27	11
8	3	28	11
9	4	29	10
10	4	30	9
11	4	31	11
12	5	32	10
13	5	33	11
14	5	34	10
15	6	35	13
16	5	36	14
17	6	37	19
18	6	38	19
19	6	39	20
20	6	40	22
21	6	41	24
22	7	42	25
23	6	43	24
24	9	44	24
25	6	45	15/6"

* No jetting.

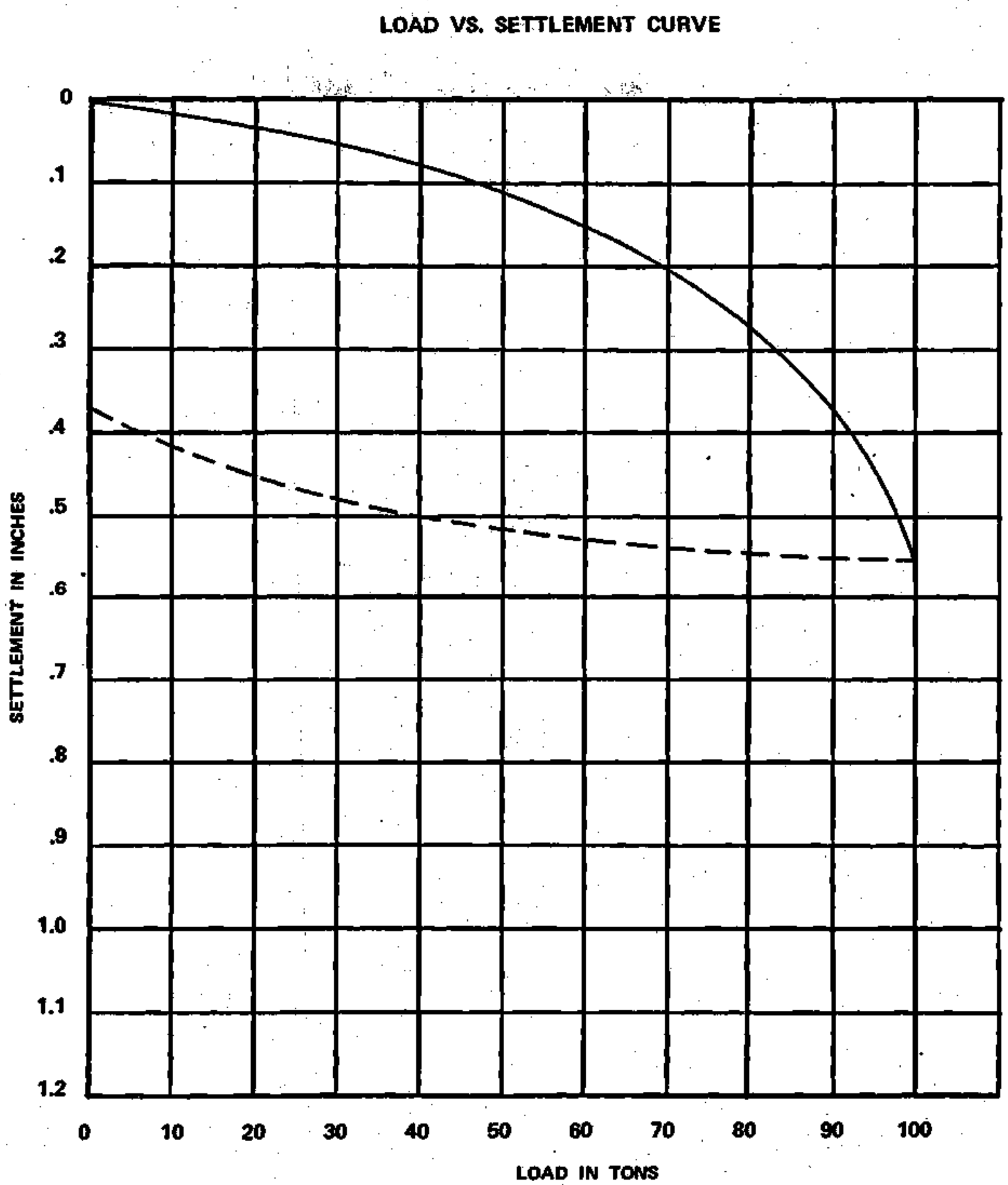

Figure 9.

Load vs. Settlement Curve Test Pile 1-1

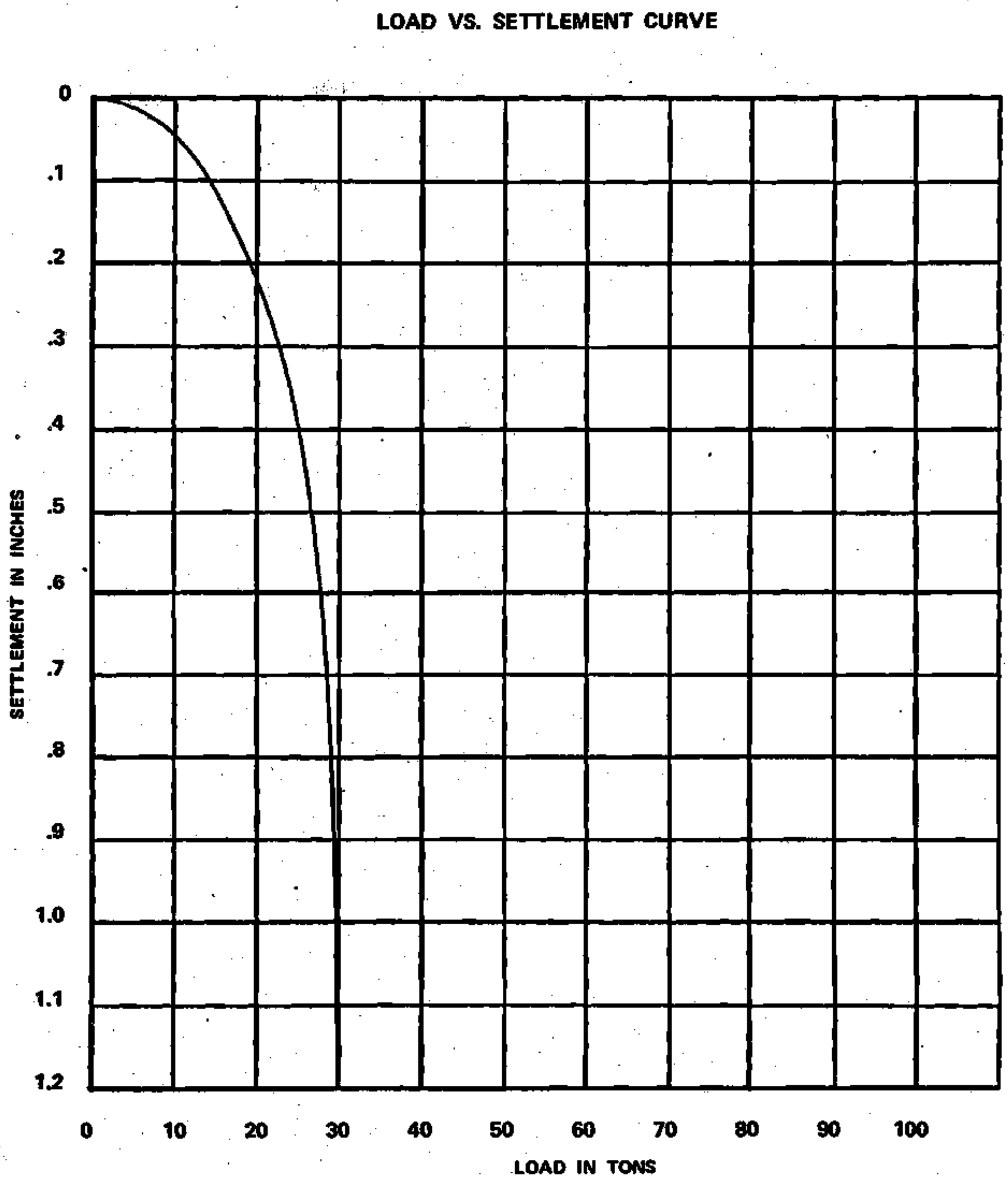

Figure 10.

Load vs. Settlement Curve Test Pile 1-3A

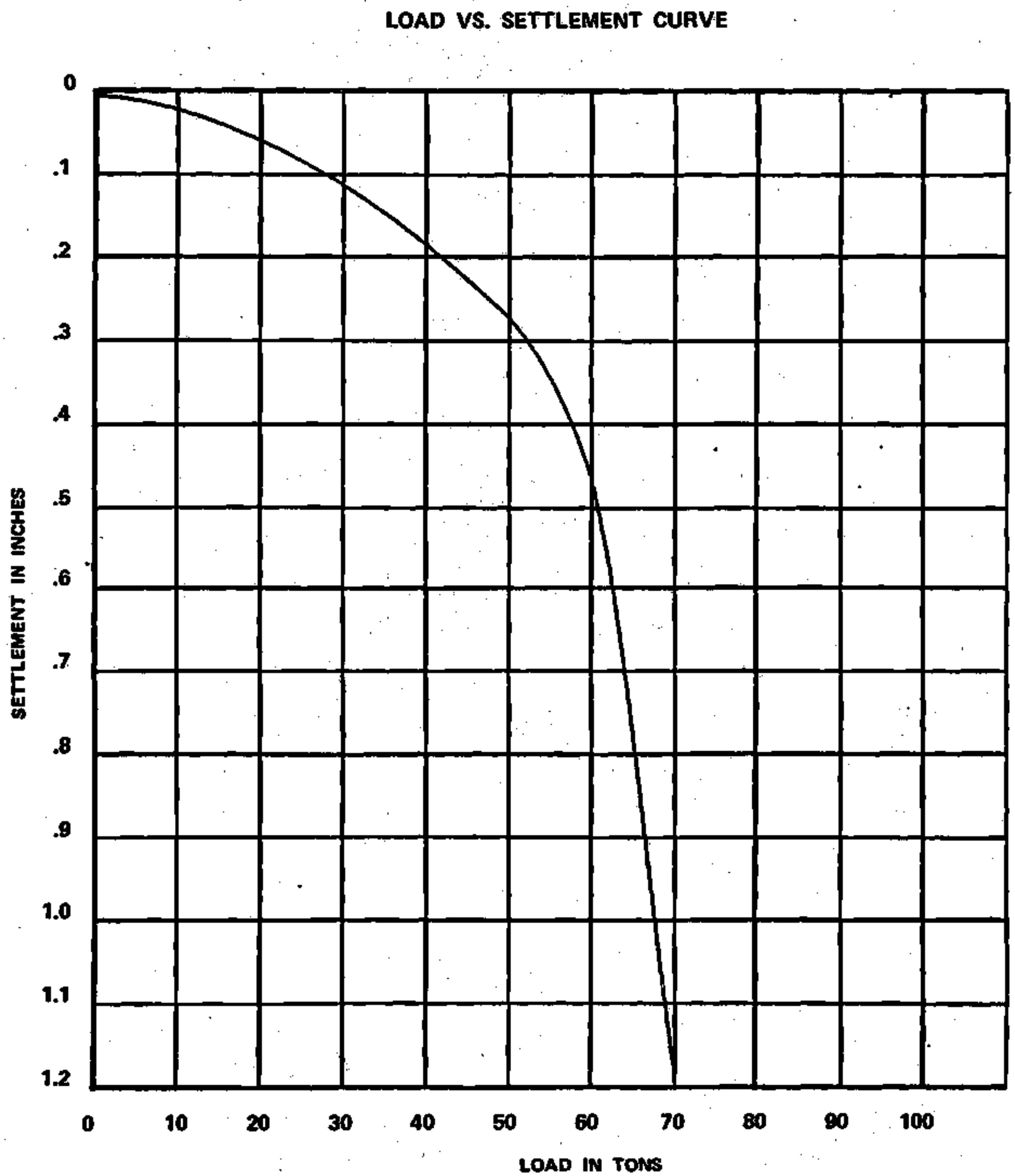

Figure 11.

Load vs. Settlement Curve Test Pile 1-4

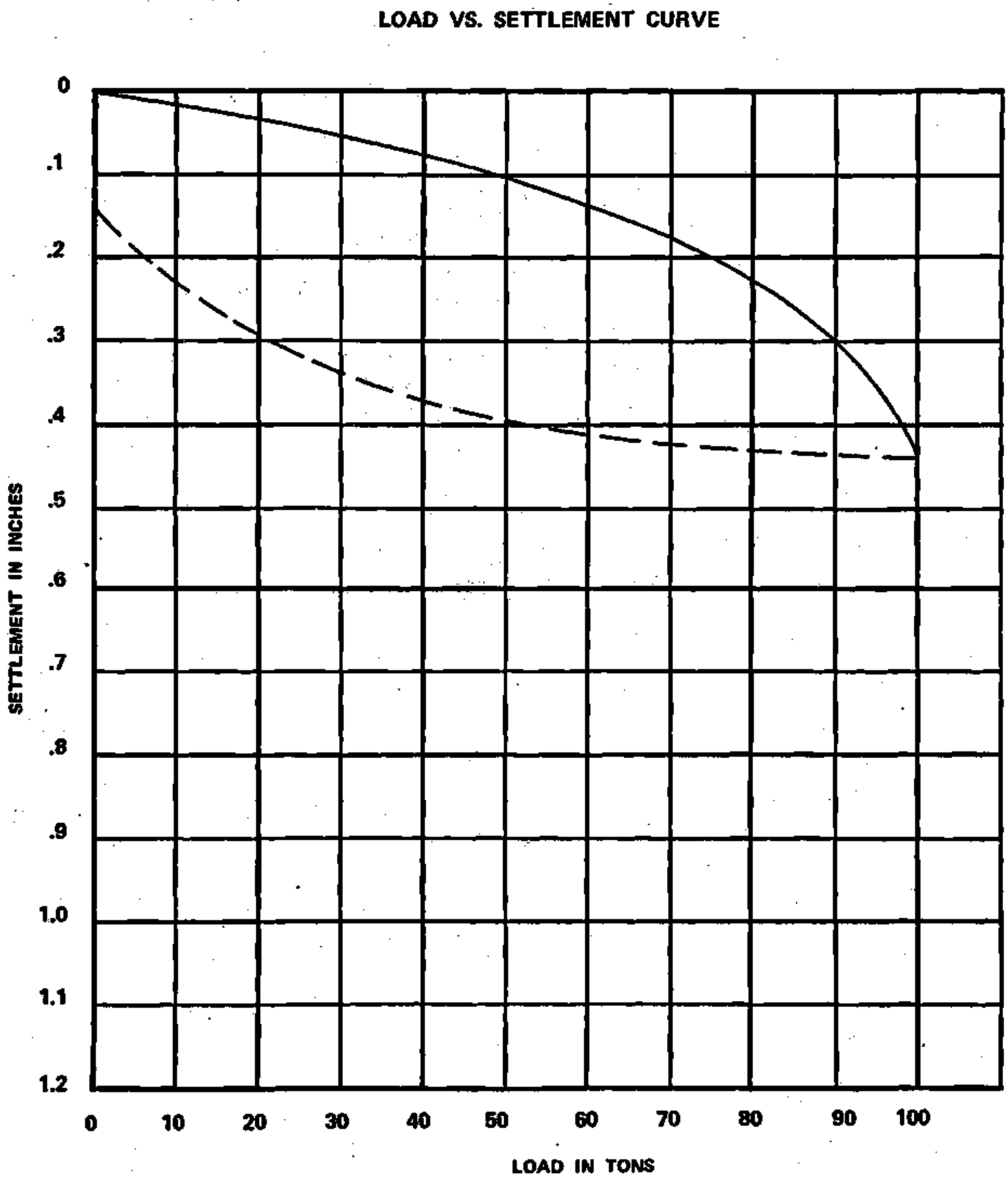

Figure 12.

Load vs. Settlement Curve Test Pile 1-5

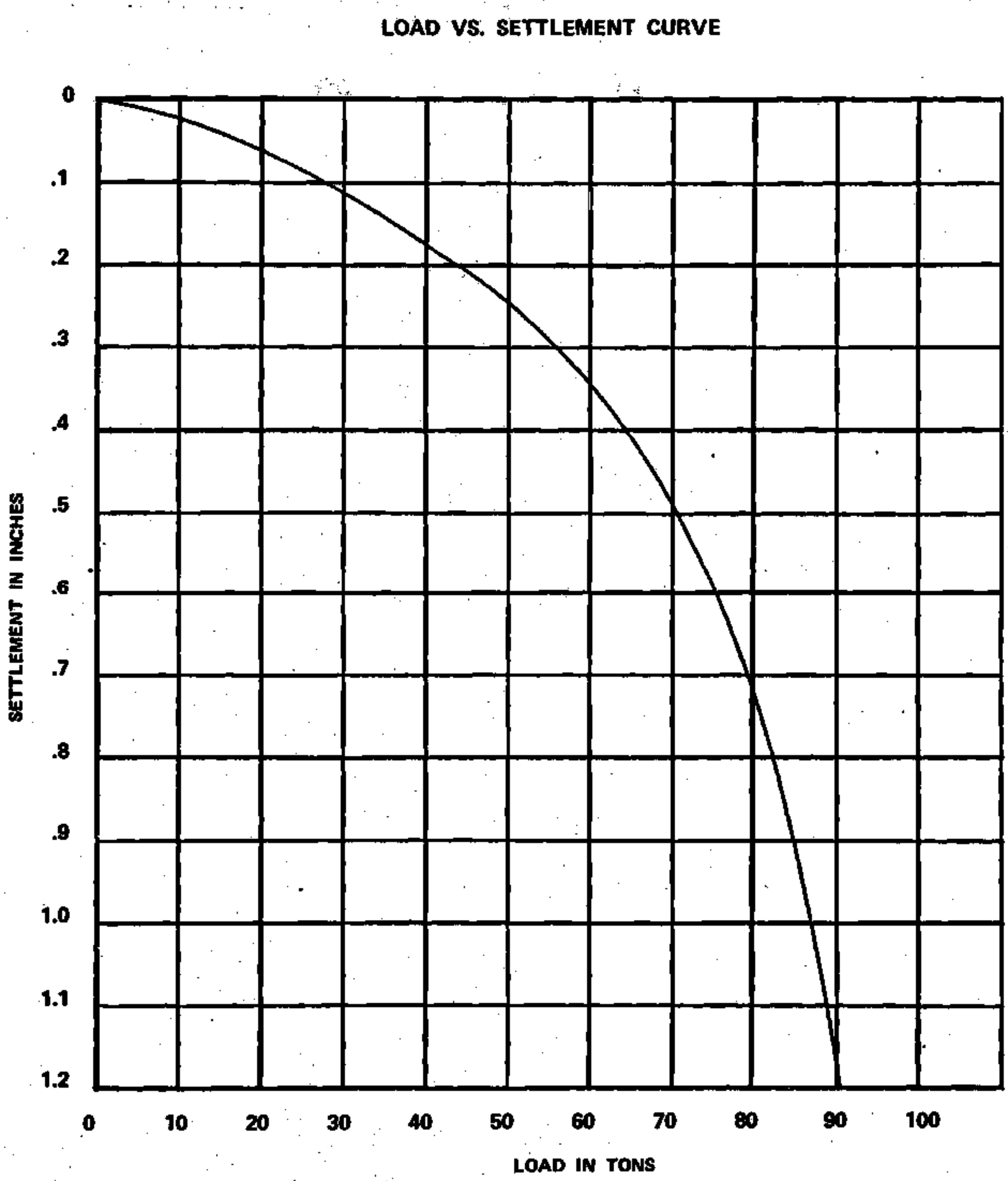

Figure 13.

Load vs. Settlement Curve Test Pile 1-6

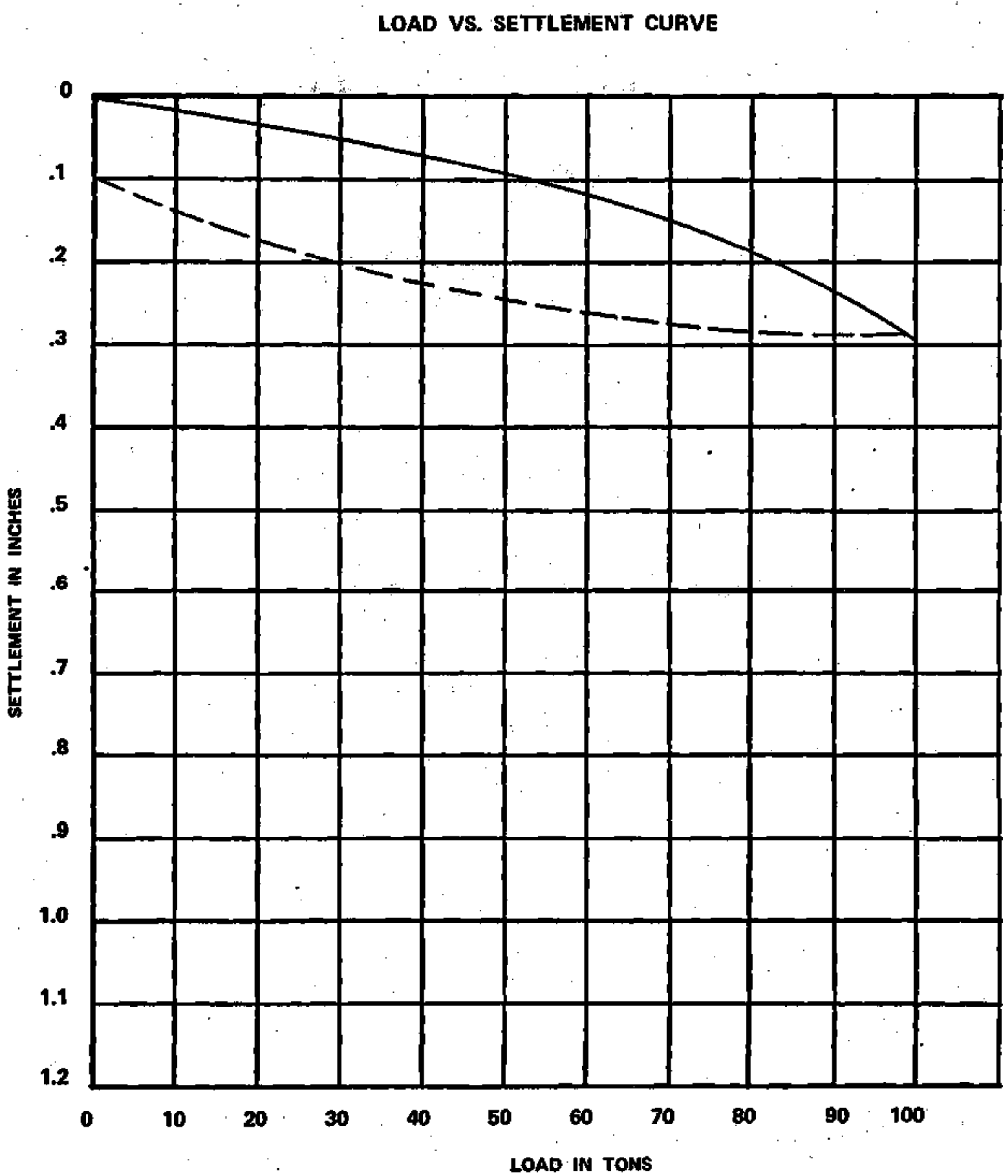

Figure 14.

Load vs. Settlement Curve Test Pile 2-1

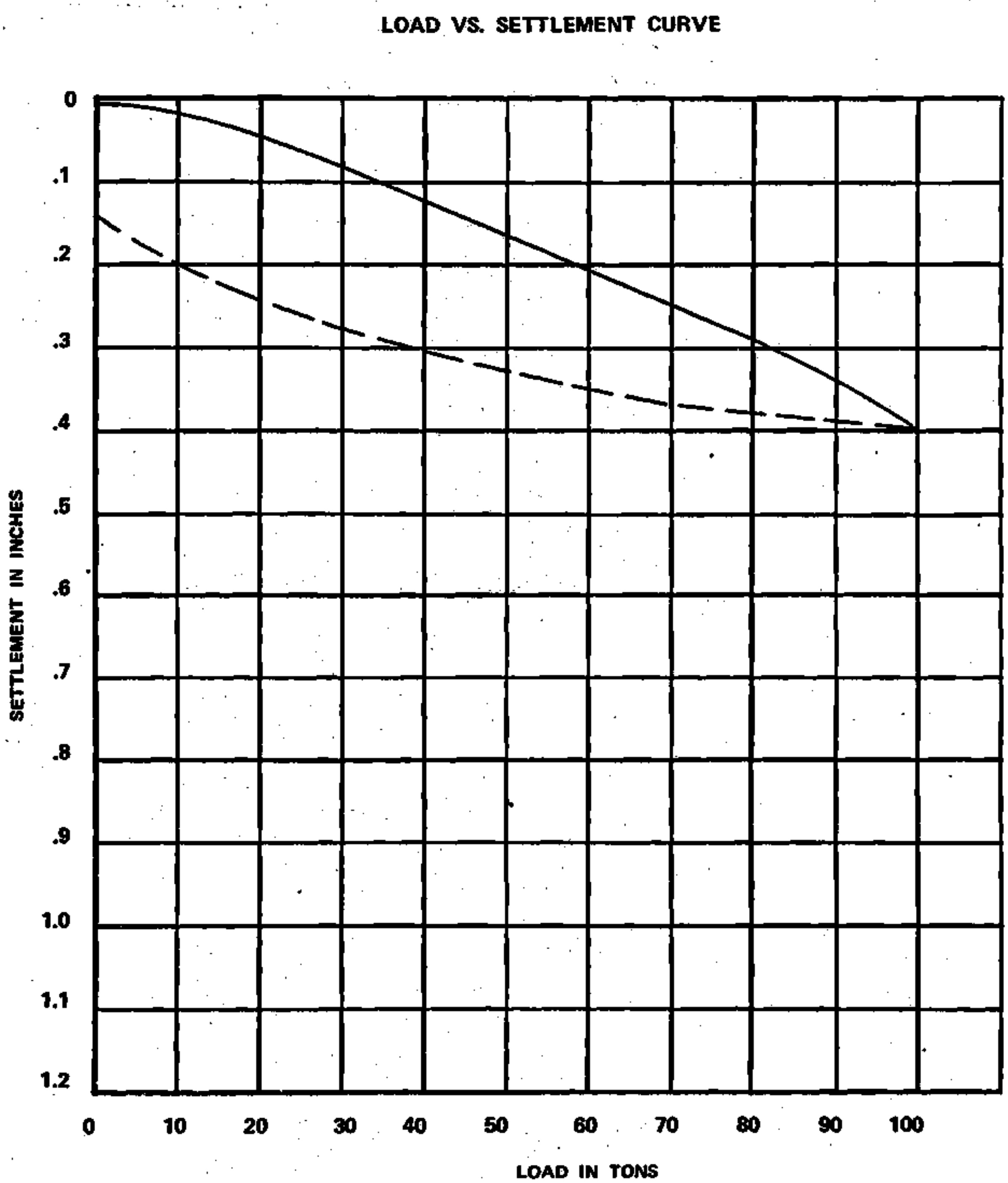

Figure 15.

Load vs. Settlement Curve Test Pile 2-2

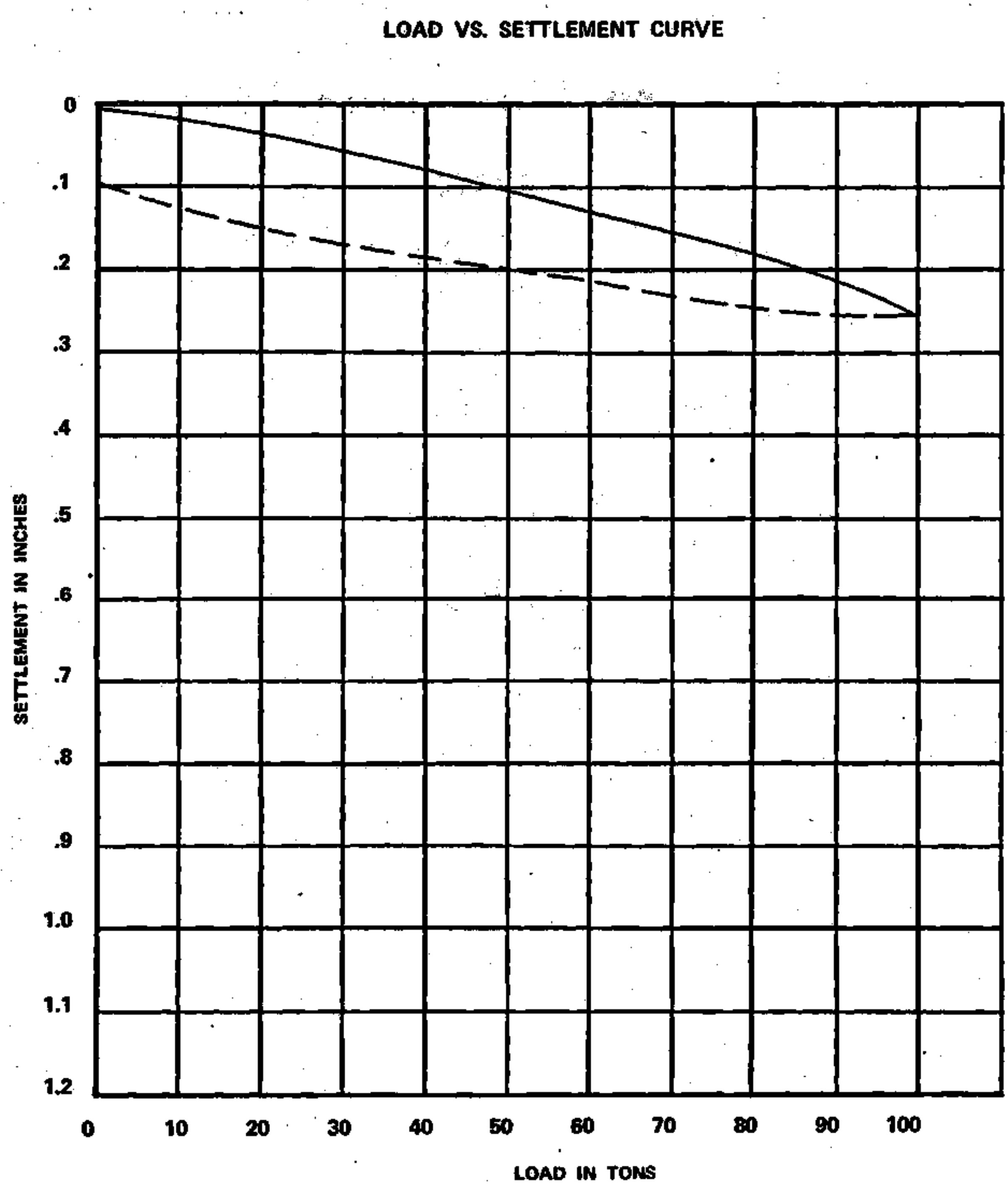

Figure 16.

Load vs. Settlement Curve Test Pile 2-3

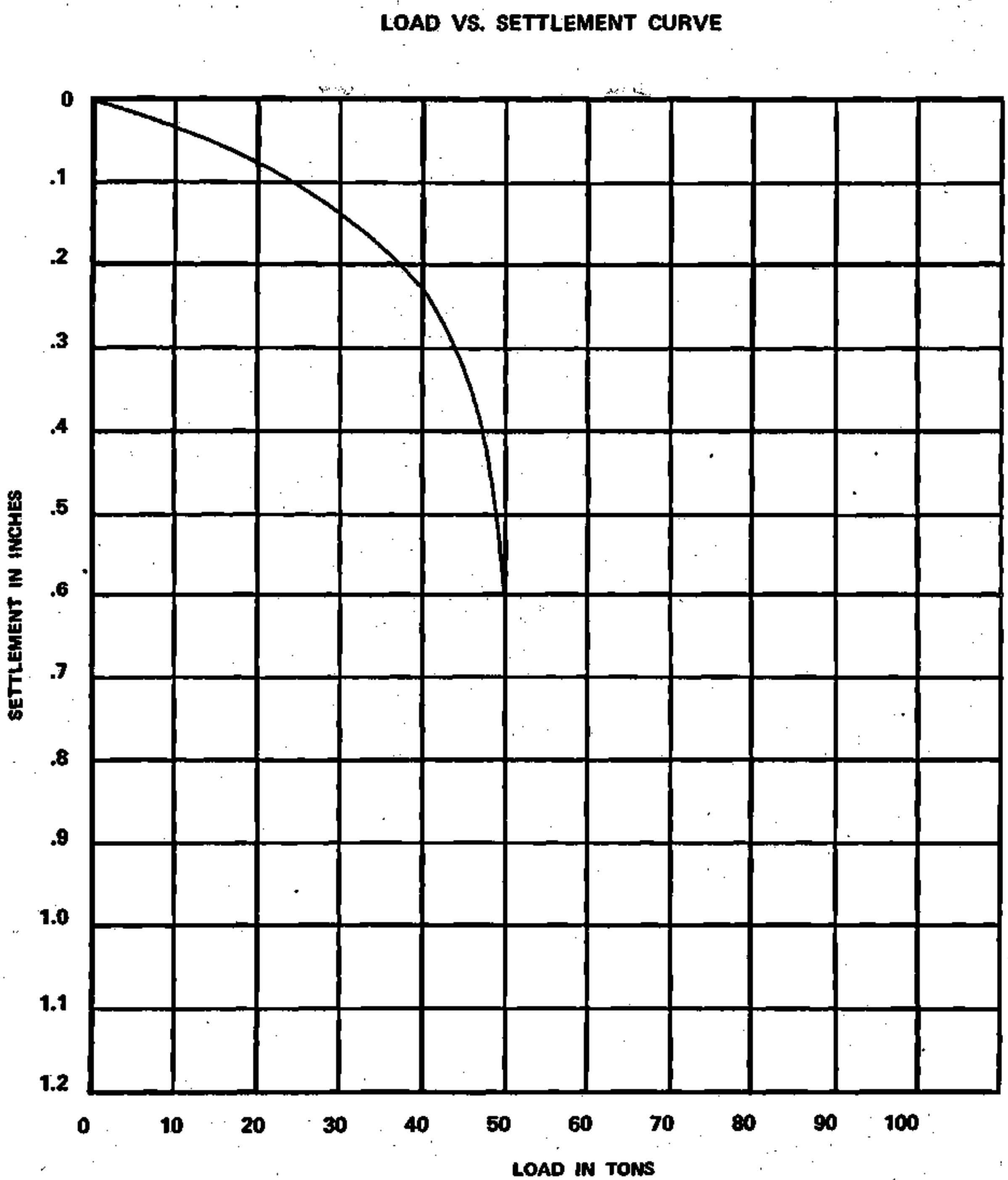

Figure 17.

Load vs. Settlement Curve Test Pile 2-4

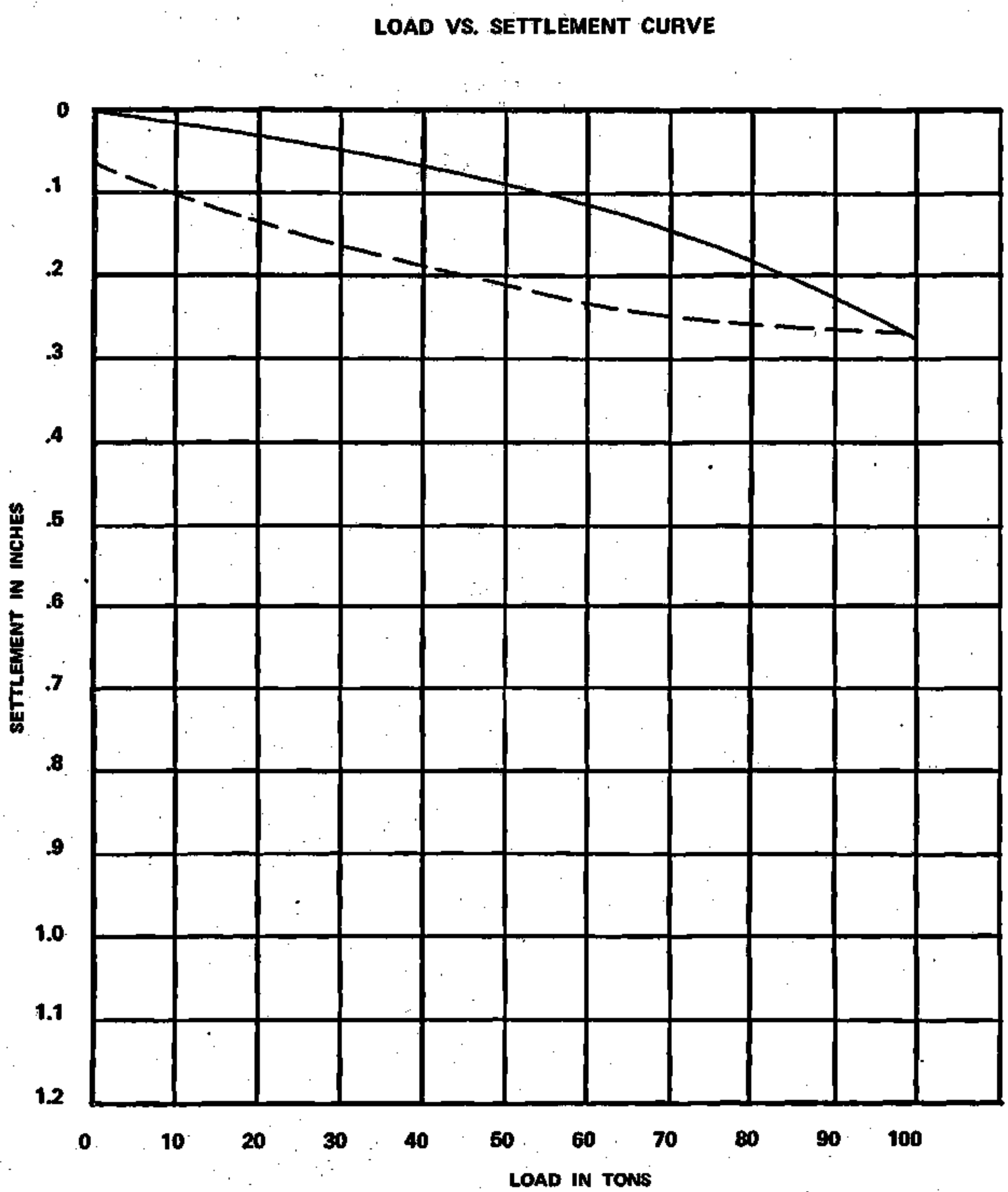

Figure 18.

Load vs. Settlement Curve Test Pile 2-5

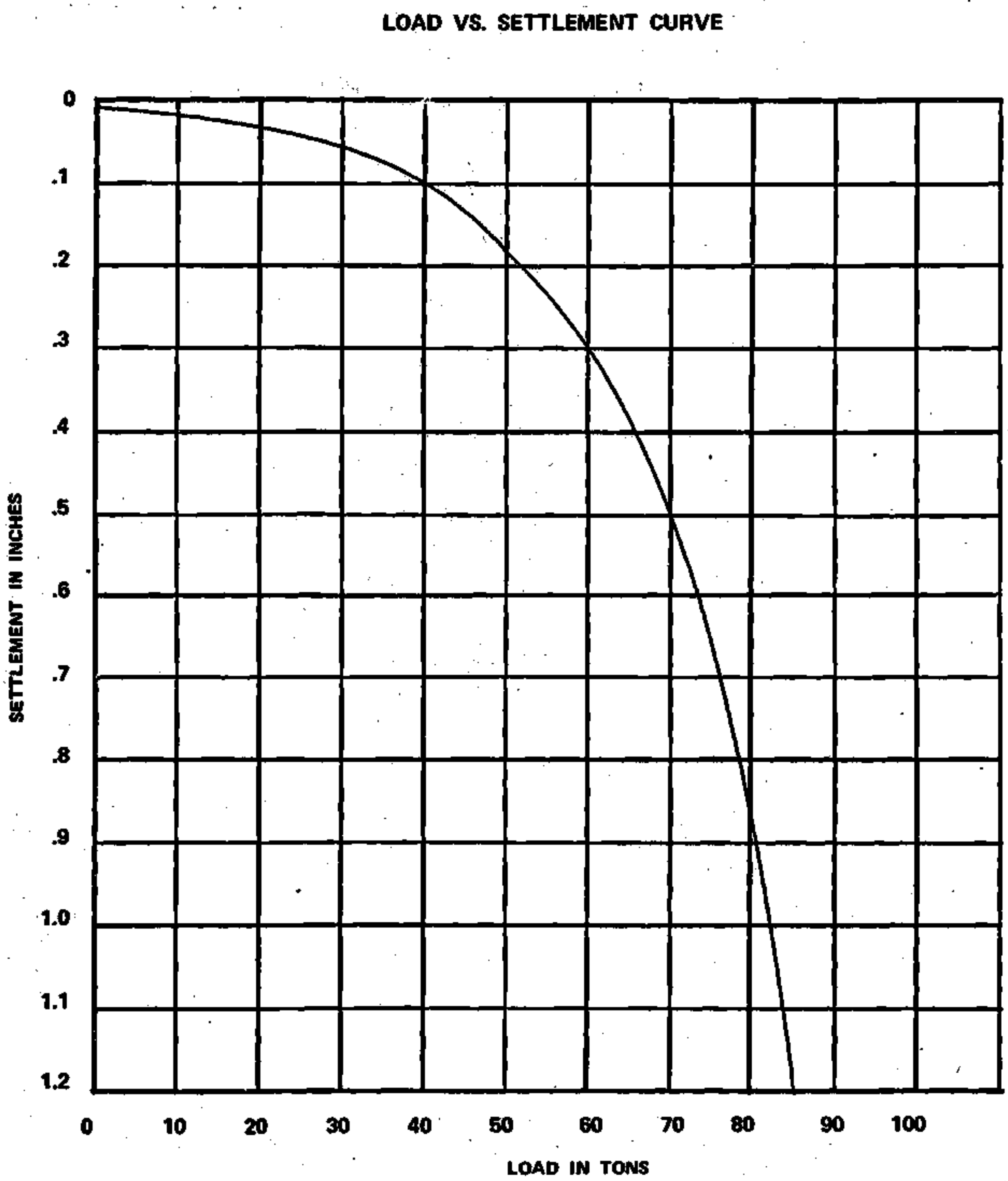

Figure 19.

Load vs. Settlement Curve Test Pile 2-6

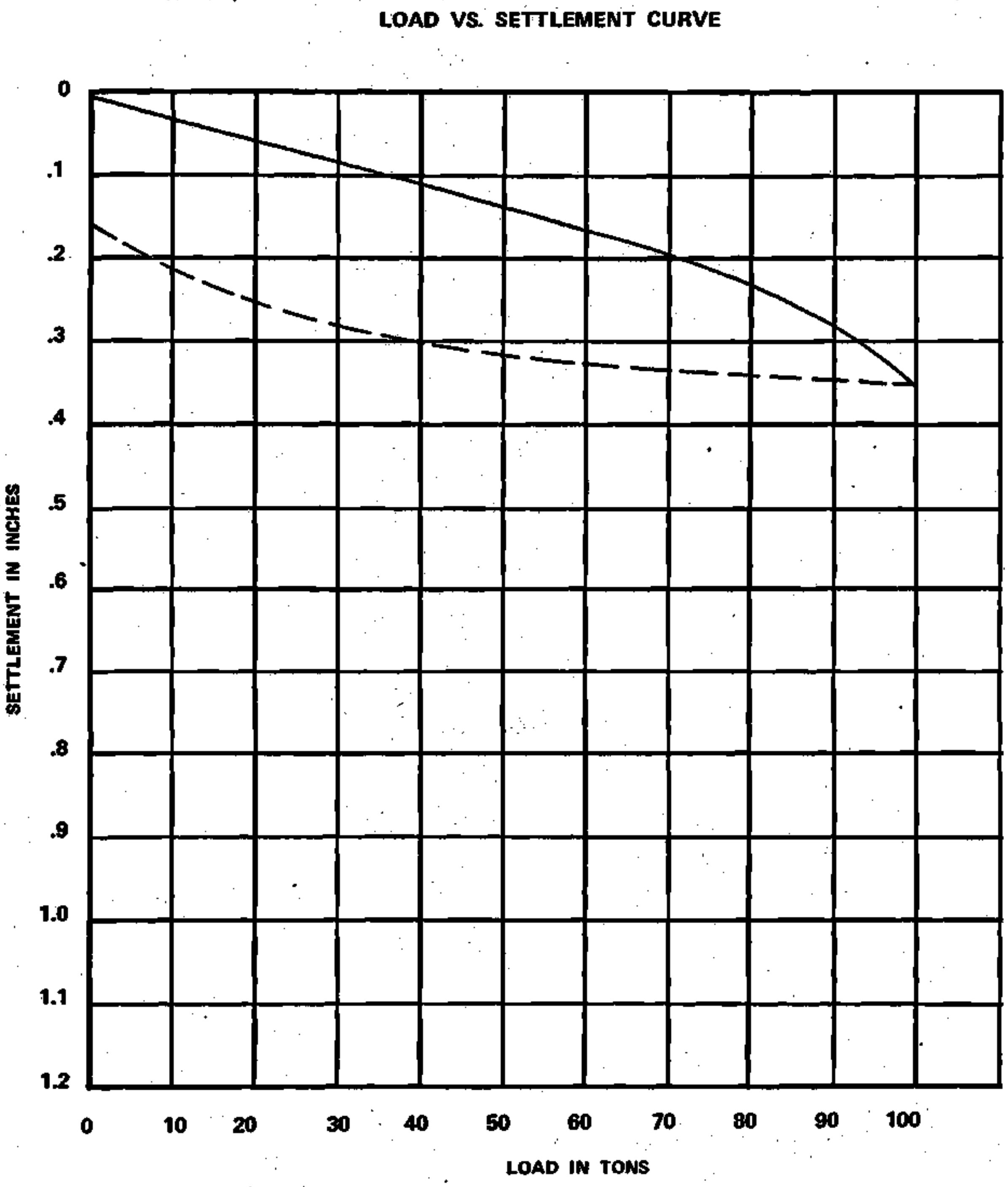

Figure 20.

Load vs. Settlement Curve Test Pile 3-1

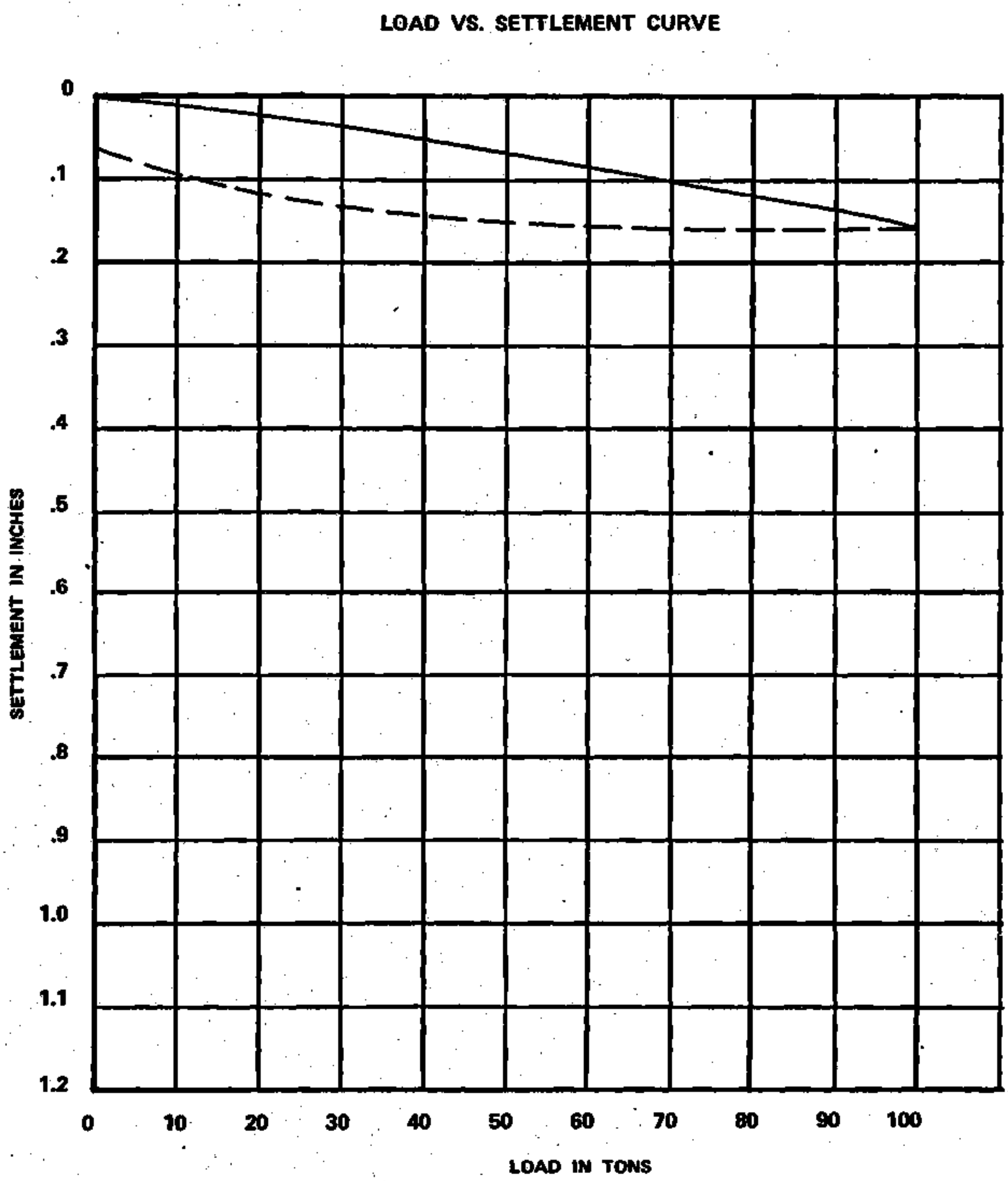

Figure 21.

Load vs. Settlement Curve Test Pile 3-2

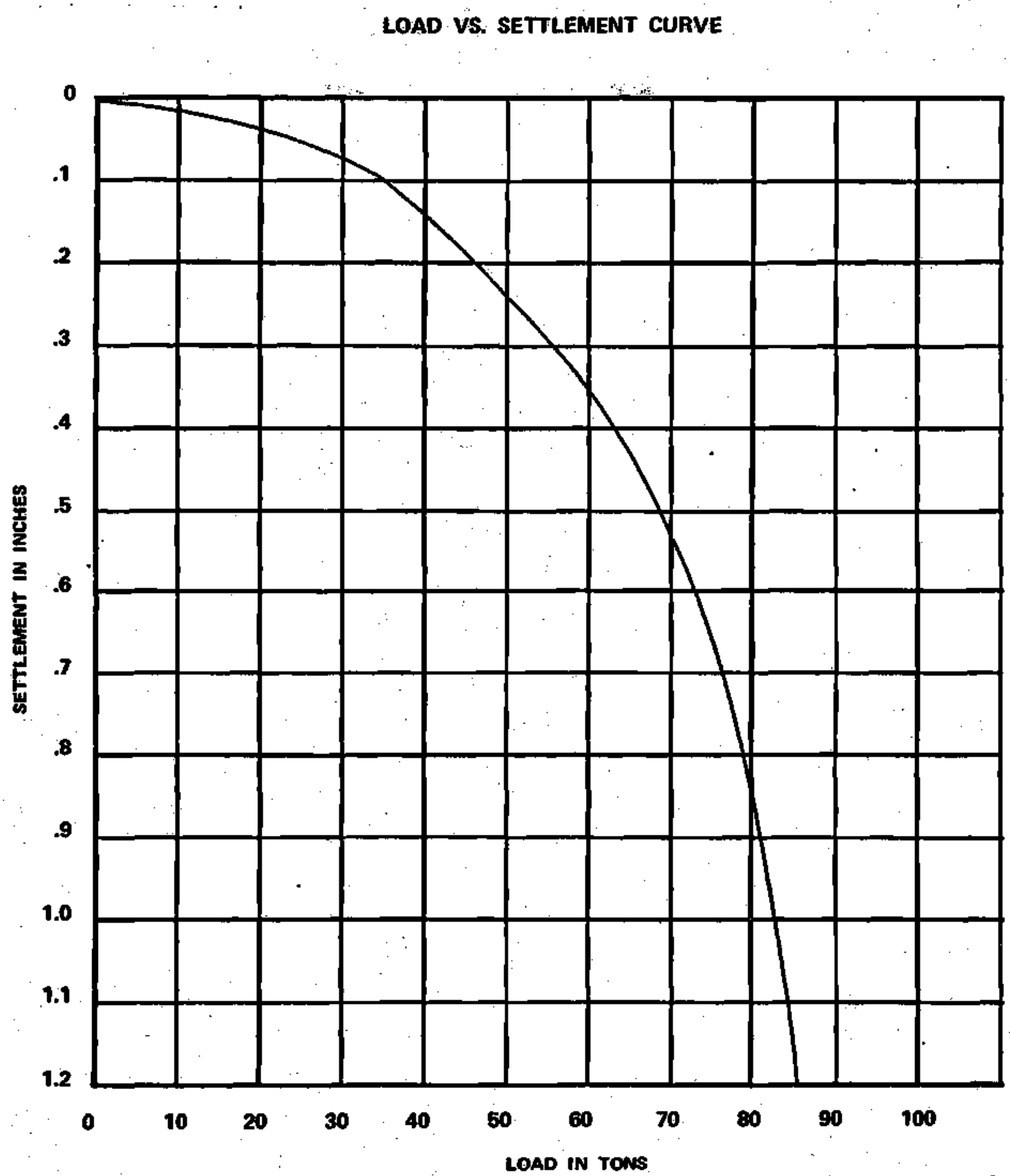

Figure 22.

Load vs. Settlement Curve Test Pile 3-3

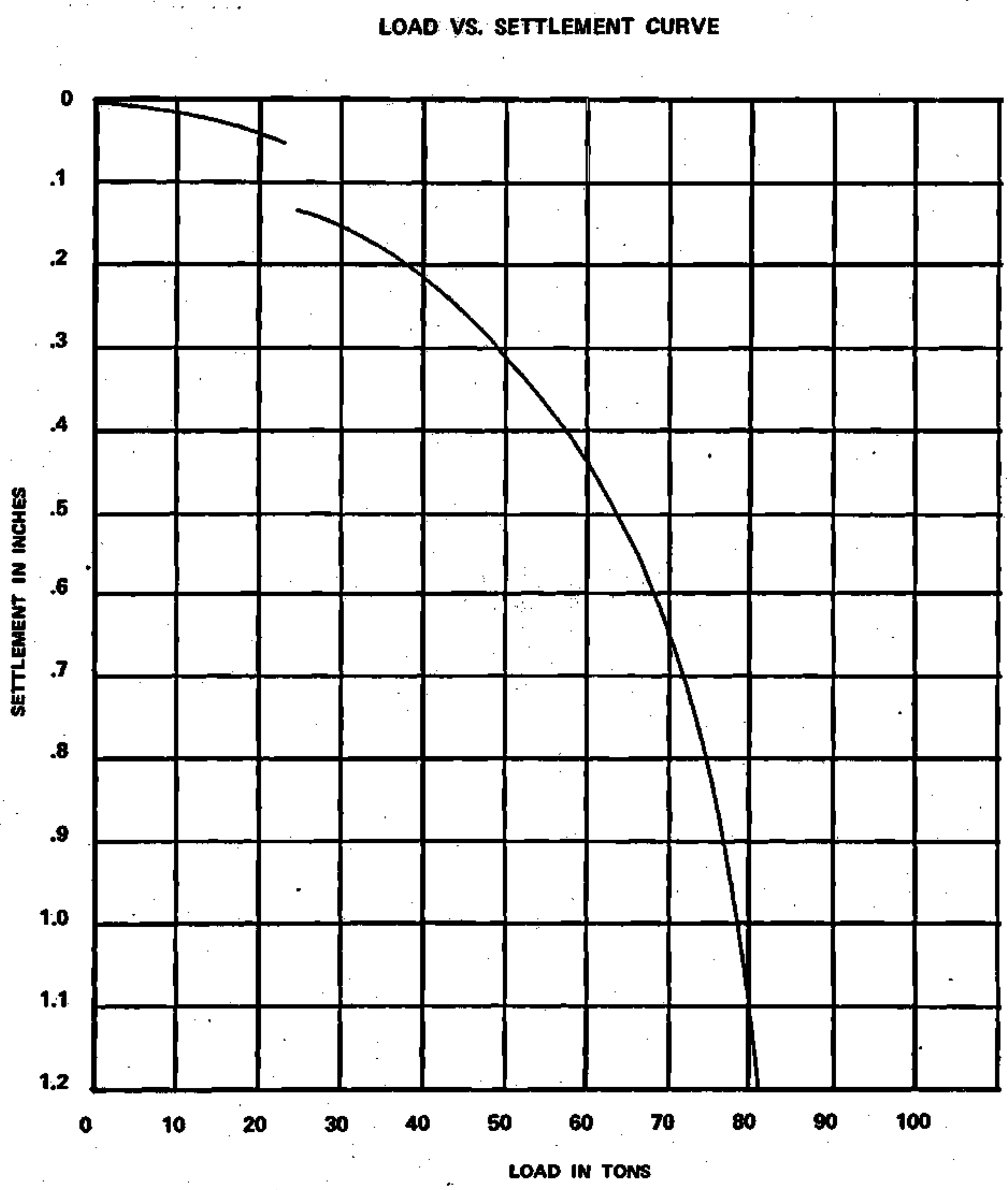

Figure 23.

Load vs. Settlement Curve Test Pile 3-4

Table 9. Test Pile Data

Jack Watson Steam Plant - Gulfport, Mississippi

Procedure

Four timber piles were driven in each of five test areas. One pile was driven from the surface to a 20 ton capacity, two were jetted to 20 feet and then driven (one to 15 tons capacity and one to 20 tons capacity), and one pile was jetted to 20 feet and not driven. The driving criteria was established by the Hiley formula. The piles were loaded to a maximum of 50 tons.

The soils at the site are similar to those at Barry Steam Plant. They consist generally of loose to very firm sands which are occassionally clayey and contain some clay lenses. Data from the program is presented on the following page. The designations are as follows:

1. Driven from surface to 20 ton capacity.
2. Jetted to 20 feet and driven to 20 tons capacity.
3. Jetted to 20 feet and driven to 15 tons capacity.
4. Jetted to 20 feet and not driven.

Table 9. Test Pile Data

Jack Watson Steam Plant - Gulfport, Mississippi

Summary

Test Area	Pile	Tip Elev.	Set (in.)	Net Deflection (in.)	E.N. Cap. (tons)	Hiley Cap. (tons)	Ratio Hiley/ E.N.
A	1	-10.1	0.09	0.016	42.5	81.8	1.92
	2	-23.2	0.11	0.431	38.6	67.3	1.74
	3	-24.2	0.30	1.169	*	*	*
	4	-20.2	-	3.282	*	*	*
B	1	-15.3	0.10	0.015	40.5	70.5	1.74
	2	-25.5	0.20	0.762	42.5	72.8	1.72
	3	-21.4	0.09	0.274	*	*	*
	4	-20.2	-		*	*	*
C	1	-15.4	0.12	0.102	36.9	65.5	1.78
	2	-23.0	0.10	0.096	40.5	69.8	1.73
	3	-24.6	0.10	0.071	*	*	*
	4	-20.3	-	1.352	*	*	*
D	1	-16.5	0.11	0.043	38.6	78.5	2.03
	2	-26.4	*	0.081	*	*	*
	3	-24.0	0.13	0.838	*	*	*
	4	-19.9	-	1.302	*	*	*
E	1	-10.5	0.02	0.031	67.5	82.6	1.23
	2	-26.6	0.09	0.082	42.5	72.0	1.70
	3	-22.8	0.10	0.038	*	*	*
	4	-20.1	-	0.245	*	*	*

* Not Analyzed
** Not Recorded

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